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Soil Mechanics and Foundations Division

PROCEEDINGS OF THE

AMERICAN SOCIETY
OF CIVIL ENGINEERS

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Proceedings of the American Society of Civil Engineers

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INFLUENCE OF GEOLOGICAL CONDITIONS ON UPLIFT

By W. Harold Stuart,[1] F. ASCE

SYNOPSIS

The selection of uplift assumptions at the base of dams has varied widely among designers. In general, the values selected for dams constructed in the past 20 yr have been conservative, as indicated by uplift observations. In several instances, uplift pressures higher than the design assumptions have been observed. A comparison is made of the geological conditions and the uplift pressures at several dams observed during this period. Though no definite correlation is established, some general guides can be drawn from the experiences described.

GENERAL

In the design of a concrete dam, the engineer is concerned with the uplift in the foundation rock and at the base of the dam, as well as the uplift that occurs within the dam itself. This examination will be restricted to the uplift that occurs in the foundation and at the base of the dam. Uplift pressures at the base of the dam and in the foundation influence the sliding factor and the stability against overturning. The designer has established uplift assumptions for determining the stability of a structure. These assumptions may vary considerably with different designers and are also influenced by geological conditions. The range of assumptions in general use are shown on Fig. 1. This

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figure shows the maximum condition as the full reservoir head acting at the heel of a dam with an assumed straight-line variation from the upstream face to zero or tailwater pressure at the toe of the dam. This may be modified by assuming these pressures to act over only a portion of the base area, or by assuming the pressures are reduced by foundation drainage to arrive at design assumptions as shown on the pressure diagram. The importance of using criteria that will provide an adequate factor of safety so that the dam will be structurally safe cannot be overemphasized. The less that is known of the foundation conditions, the more conservative these criteria must be. As an illustration of how the criteria for uplift may affect the economy of the dam, assuming all other conditions as remaining constant, if 100% uplift is assumed, a base width to height ratio of 0.84 is required; for 67%, the ratio of base width to height is 0.76; and for 50%, the ratio is 0.72. In a high dam the difference in cost of the different sections could easily run into millions of dollars. Foundation treatment by grouting and drainage must be considered along with geological conditions in determining the uplift criteria. One of the fundamental principles of design requires that the upstream portion of the foundation be as tight as possible and that the escape of such water as might pass the upstream heel of the dam be made as favorable as possible.

A detailed knowledge of the geological conditions, along with a well designed program of foundation treatment, grouting, and drainage, is required to construct an economical and safe foundation. There are many geological factors that influence the uplift in a foundation and at the base of a dam. Some of these are the type, structure, condition, and topography of the foundation rock. The pattern and direction of channels, fractures, and seams, and the extent that they are filled with gouge, mineral deposits, and weathering products, will affect the uplift acting at the base of the dam and in the foundation. In sedimentary rocks where impervious beds are interbedded with open pervious beds, the situation may be either favorable or detrimental to the relief of uplift pressures. If in the sedimentary strata the dip is upstream and the dam is founded on open free draining rock, a tight grout curtain located near the heel of the dam tied into an impervious lower bed will cut off the reservoir head effectively, so that only the uplift from tailwater is effective downstream from the grout curtain (Fig. 2). Fig. 2 shows a concrete dam built on a fractured sandstone foundation underlain by tight shale. The dip of the strata is upstream, and the contact of the sandstone and shale is exposed downstream from the dam. The sandstone is free-draining, and the contact of the sandstone and shale is open. The sandstone is susceptible to the construction of a tight grout curtain which can be tied into the shale that underlies it. The free-draining characteristics of the sandstone, along with the drainage holes immediately downstream from the grout curtain, result in reduction of uplift pressures to that imposed by the tailwater immediately downstream from the grout curtain. In contrast, if the beds dipped downstream, an impervious stratum near the foundation, underlain by an open pervious zone, may trap the head of the reservoir so that the maximum uplift imposed by the reservoir will act on a large portion of the base of the dam or on the foundation rock. Grouting of the open horizon will not alleviate this condition because the grout curtain can be bypassed. Excessive uplift pressures will obtain downstream from the grout curtain unless an efficient drainage system is installed to collect all water that may bypass the grout curtain.

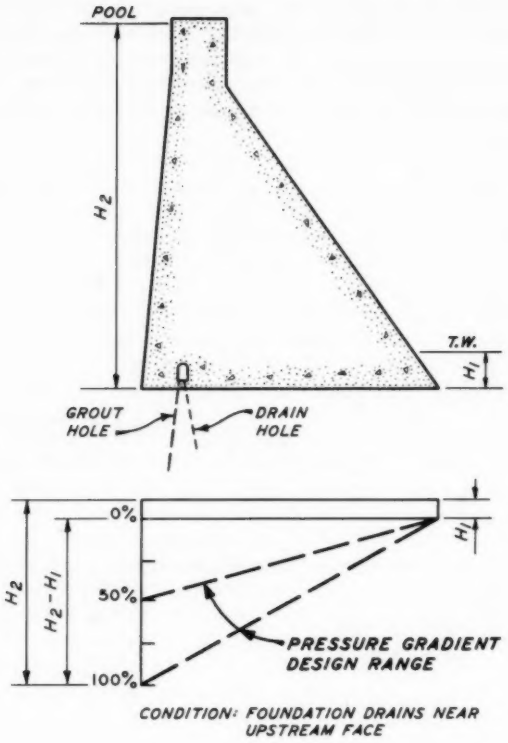


FIG. 1.—UPLIFT PRESSURE DIAGRAM

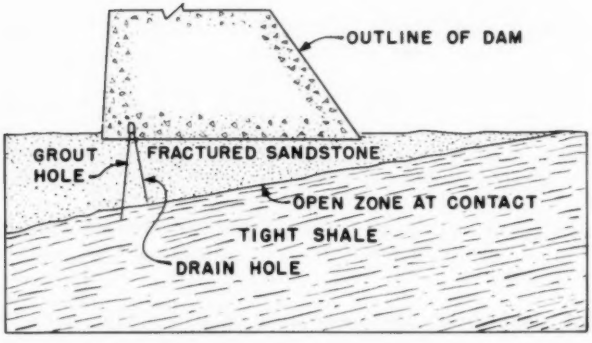


FIG. 2.—CONCRETE DAM BUILT ON A FRACTURED SANDSTONE FOUNDATION UNDERLAIN BY TIGHT SHALE

Fig. 3 illustrates a condition that may exist where jointed and fractured basalt flows and ancient slots or potholes have been cut through the basalts downstream from the structure. In addition to the openness of the joints and fractures, the flow contacts of the successive basalt flows are open and free draining, and the pothole or slot affords additional relief. With a grout curtain

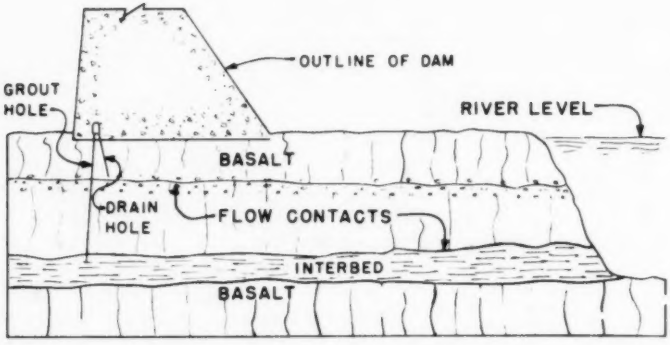


FIG. 3.—CONDITION EXISTING WHEN JOINTED AND FRACTURED BASALT FLOWS AND ANCIENT SLOTS OR POTHOLES HAVE BEEN CUT THROUGH THE BASALTS DOWNSTREAM FROM THE STRUCTURE

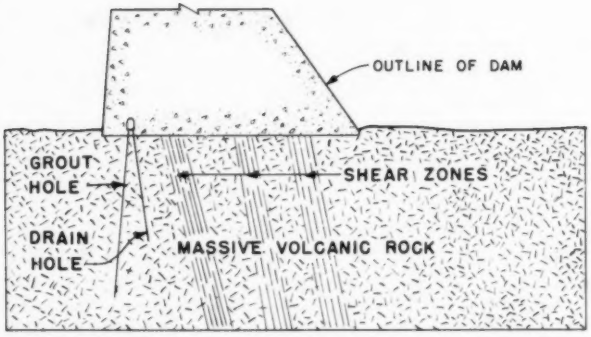


FIG. 4.—DAM BUILT ON A FOUNDATION OF MASSIVE VOLCANIC ROCK

and drainage holes near the upstream face of the dam, in this case, we can expect excellent relief of uplift pressures through the fractures and the flow contacts, so that uplift pressure downstream from the drainage holes would normally be equal to tailwater.

Another situation is the foundation in massive igneous or volcanic rock with tight fractures and seams. The efficiency of the grouting and drainage in such foundations is difficult to evaluate, because the drainage is dependent on the intersection of the fractures and joints by the drainage holes. Foundations in these materials may be further complicated by shear zones and faulting, which in some cases may work to the advantage of reducing uplift pressures but in other cases will be detrimental. For example, Fig. 4 illustrates a dam built on a foundation of massive volcanic rock that is cut by shear zones essentially parallel to the axis of the dam. These shear zones may occupy a large portion of the foundation area, and because of the massiveness and tight character of the rock in the zone of grouting and drainage, an effective job of relief from uplift is difficult to accomplish. Water that bypasses the grout curtain and the drainage system collects in or adjacent to these shear zones and may exert considerable uplift against the base of the dam, at pressures greater than assumed in the uniform pressure diagram. It is possible also in this type of rock that the shear zone or gouge zone may occur at a location favorable to reducing the leakage into the foundation area. The shear zone may be essentially vertical and be filled with tight clayey gouge located so that it complements the grout curtain. This will improve the effectiveness of the grout curtain and reduce the amount of seepage that must be intercepted by the drainage holes.

In the past thirty years considerable data have been collected regarding uplift pressures under dams, some of which have been published in construction magazines and technical publications. Much of the published data indicate that, in general, uplift pressures as observed are less than the accepted design assumptions. However, there are observations that are rather alarming, in that the uplift pressures observed were much higher than those assumed in the design. Design assumptions, foundation treatments, uplift observations, and corrective treatment, performed as required to maintain design uplift assumptions for several dams constructed by the Corps of Engineers on foundations of varying geological characters, will be described in the following paragraphs.

NIMROD DAM

Nimrod Dam is a concrete gravity dam on the Fouche La Fave River in Arkansas, with a hydraulic height of 62 ft. (Hydraulic height as used herein is the difference between normal tailwater and maximum reservoir pool.) The design assumption for uplift was full reservoir pressure at the heel of the dam, with a straight-line variation to the tailwater pressure at the toe acting over two-thirds of the base area. The foundation materials consist of highly fractured quartzitic sandstone underlain by a tight shale with an upstream dip as illustrated by Fig. 2. The grout curtain consists of holes on 5-ft centers that extend to an average depth of 75 ft through the sandstone-shale contact. The drainage system consists of 3 in. diameter holes at 10 ft centers; these were also drilled through the shale-sandstone contacts in the river section. During construction artesian flows were noted in the foundation area. The shale and sandstone contact also dips towards the left abutment, and it appeared that the main source of this artesian water was from the right abutment. Three years after the dam had been put into operation, flow and pressure measurements were made along the line of drainage holes just downstream from the grout

curtain. At the time of these measurements, both the reservoir and tailwater were varied. Fluctuation of the reservoir pool had no apparent effect on the uplift pressure, but fluctuations of the tailwater were reflected almost immediately. The maximum head measured was slightly less than tailwater. It was assumed that the most direct connection between tailwater and drainage holes was along the sandstone shale contact which is exposed in the riverbed about 600 ft downstream. However, the sandstone is also highly fractured and may offer direct connection with the tailwater. In 1952, after ten years of operation, the uplift pressures were again observed with low tailwater conditions. The drainage holes had been capped for some time and the maximum pressure observed was 8 psi. This pressure was somewhat greater than what would be caused by tailwater. After the caps had been removed from the holes to make discharge measurements and the holes had been recapped, the maximum pressure observed was 5 psi. When the adjacent hole was uncapped, the pressure of the hole being observed was immediately lowered. The measured pressure for few holes, which were slightly greater than would obtain from tailwater, suggested artesian pressure from the abutments or inefficient grout curtain in a localized area. This foundation has an efficient grout curtain, an artificial drainage system, and natural drainage. With the artificial drainage inoperative, the maximum uplift was only about 30% of the design uplift assumption. At the time that the first pressure measurements were made, three years after the dam went into operation, the drainage holes were flushed, and considerable calcium carbonate particles were washed from the holes. It is not known whether this material resulted from leaching of the concrete or from calcite veins in the sandstone. One definite conclusion that was reached, after these observations and further study, was that the drainage holes had been drilled too deep. Had they been drilled to a depth of only about one-third of that of the grout curtain they would have performed an efficient drainage operation and reduction of uplift with much less inflow into the galleries. With the drainage holes drilled through the sandstone and shale contact, which carried a considerable quantity of water, it was difficult to keep the gallery unwatered with all of the drainage holes operating.

BULL SHOALS DAM

Bull Shoals Dam is a concrete gravity structure located on the White River in Arkansas, with a hydraulic height of 190 ft, and founded on a fractured and jointed dolomitic limestone with essentially horizontal bedding planes. The uplift criteria for design was for full reservoir pressure at the heel, with a straight-line variation to zero or tailwater at the downstream toe acting over two-thirds of the base area. A grout curtain with holes on 6-ft centers to an average depth of 80 ft was provided near the upstream face. These holes terminated in a relatively impervious stratum. Drainage holes were drilled immediately downstream from the grout curtain and were designed for a spacing of 8 ft to 9 ft and an average depth of about 60 ft. During construction it was decided that 16-ft to 18-ft spacing was adequate. In 1952, after the reservoir was placed in operation, uplift pressures as high as 50% of the design uplift assumption were observed in several monoliths at the top of the drainage holes when they were plugged. Additional drainage holes drilled in these monoliths reduced the observed uplift pressures to 20% or less of the design assumption.

Uplift pressure observations in 1953, at the top of drain holes, indicated maximums of about 35% of the design assumptions in two monoliths. Lower pressures were observed in the remaining monoliths. Apparently, the fractured and jointed foundation rock provides adequate drainage to preclude a build-up of uplift pressures even though the drain holes are on 16-ft to 18-ft centers in most monoliths.

CHIEF JOSEPH DAM

The Chief Joseph Dam is located on the Columbia River and is a concrete gravity structure with a hydraulic height of 185 ft. The foundation is predominantly granitic rocks consisting of massive granite, granite gneisses, and granite schists. The joint pattern is random with dips of 30° to 80°. Numerous faults cross the foundation area, most of which are essentially normal to the axis of the dam. They range from low to high angle faults with dips varying from 25° to 90°. Zones of crushed material are present along the fault zones ranging in thickness from 1 ft to 10 ft. In general, the foundation as prepared for placement of concrete is classed as excellent. At a few locations the crushed material along the fault zones was removed to several feet below the general foundation level and backfilled with concrete. Grouting and drainage were accomplished from a gallery near the base of the dam, with grout holes oriented in a general upstream direction and at an angle that would intersect the maximum number of joints and fractures. Drainage holes were directed in a downstream direction and were spaced on 20-ft centers. Piezometers for observing uplift pressures at the base of the dam were installed along two lines normal to the axis of the dam; one line is in monolith 11 and consists of six observation points. The other line is in monolith 19 and consists of five observation points. Additional observation holes were drilled in the plugged diversion sluices after the dam was put into operation. In monolith 11, the foundation rock consists of granite in the upstream 80% of foundation area, and the remainder is granite gneiss. One fault essentially normal to the axis of the dam crosses the foundation of monolith 11. The foundation for monolith 19 consists of granite gneiss in the upstream 80% of the foundation area, with blocky granite in the remainder of the foundation area. Two faults cross the foundation of this monolith, one with an almost vertical dip crosses the foundation in the vicinity of the 18 and 19 monolith joints near the axis of the dam, and the other crosses the downstream third point of the foundation area at an angle of about 30° with the axis of the dam. Design uplift assumptions provide for full hydrostatic pressure varying from the reservoir pool to tailwater over two-thirds of the base area. In general, all observations have indicated that uplift pressures at the base of this dam are considerably below the design assumptions as shown by Fig. 5. During the high water period of 1957, uplift pressures under monolith 19 near the upstream face exceeded the design assumptions, and at the same time the flow from the drain holes in this monolith increased to eight times that observed in 1956. In the spring of 1958, two additional drainage holes were drilled in monolith 19, both of which intercepted high pressures at the contact of the concrete and bedrock. Uplift pressures were immediately reduced to about 50% of that previously observed. During 1959, the uplift pressures remained within the design criteria; however, in 1960 they again increased until they exceeded the design assumptions again in monolith 19. In

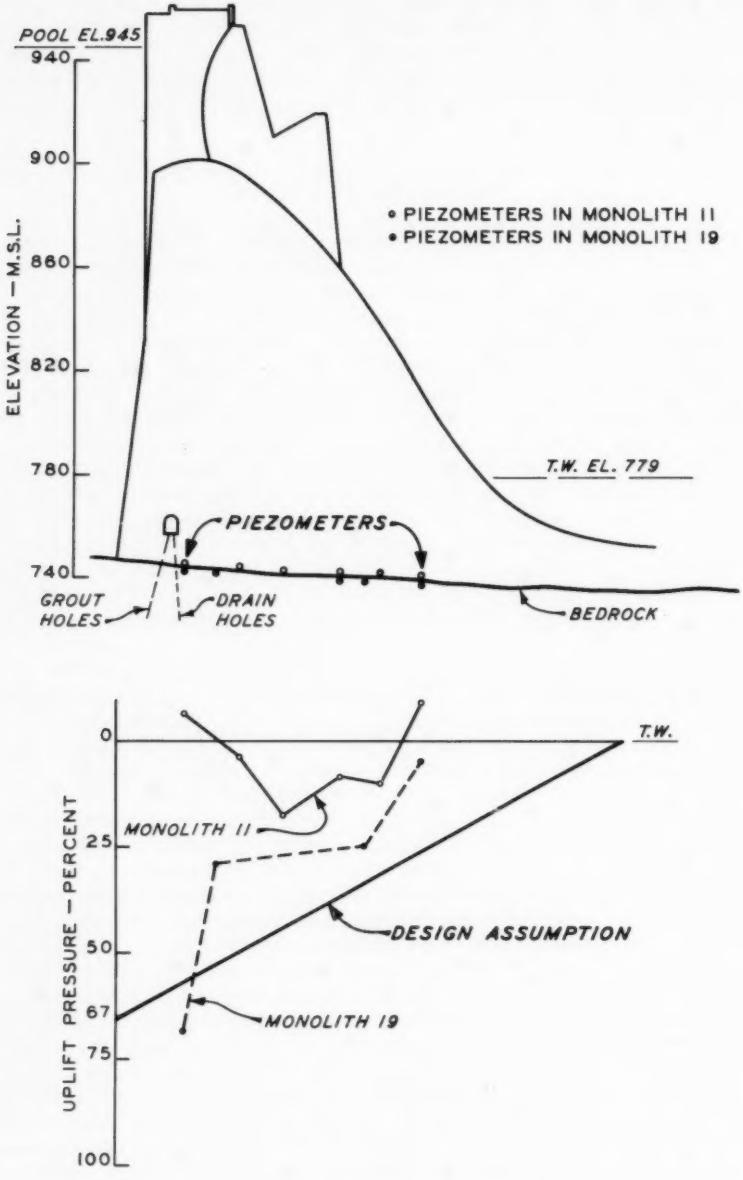


FIG. 5.—UPLIFT PRESSURES, CHIEF JOSEPH DAM

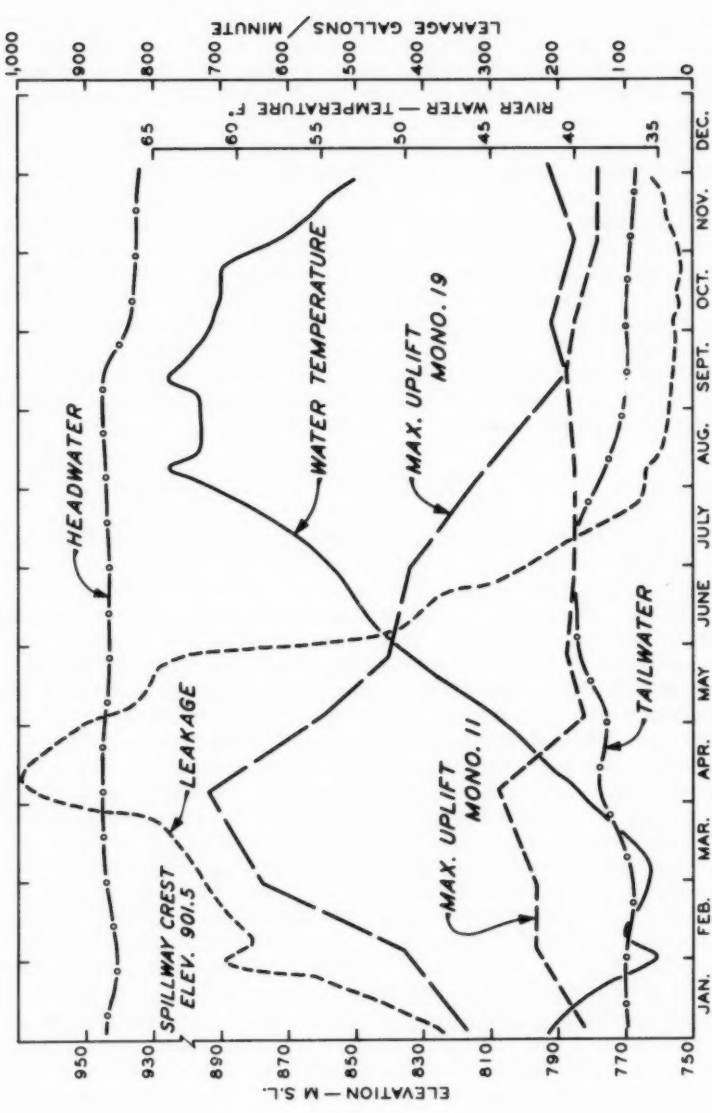
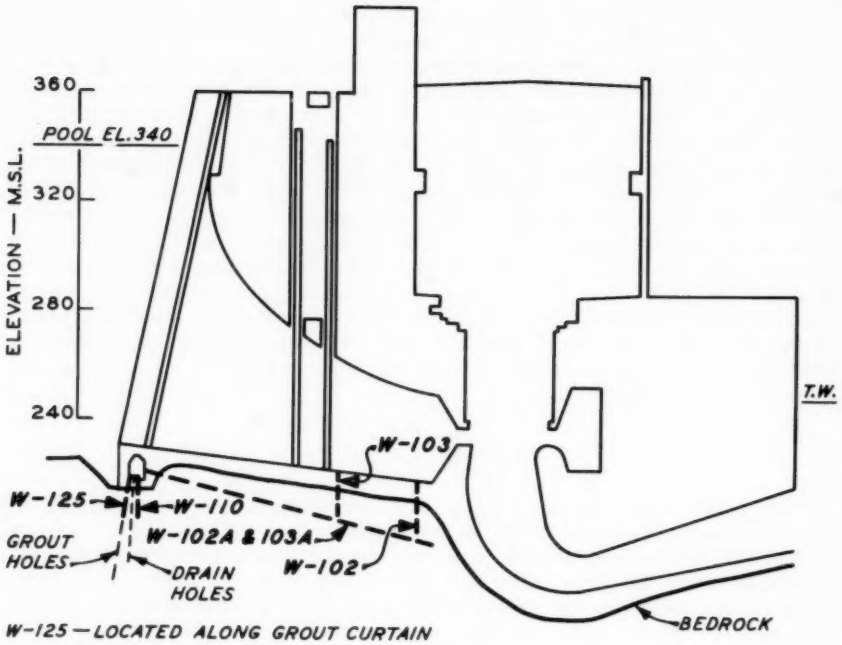


FIG. 6.—CHIEF JOSEPH DAM – 1960

January, 1961, a program of additional drain hole drilling in monolith 19 and cleaning of drain holes for the entire dam was initiated. The first two drain holes drilled were on the opposite side of the original drain hole from the hole drilled in 1958 that reduced the pressure. These holes did not reduce the uplift pressure. The third hole drilled split the spacing between the original hole and the new hole drilled in 1958. This hole relieved the pressure completely when it penetrated the concrete-foundation rock contact. Fig. 6 is an interesting illustration of the relationship of water temperature, uplift pressures, and leakage for a typical one year period. It is noted that both leakage and uplift pressures increase measurably during the period of lowest water temperatures and continue to their peak approximately one month after the temperature of the water has started rising from its low during March. The major portion of the leakage is from joints in the dam rather than from drainage holes. Cleaning of the drainage holes revealed that many of the holes were completely plugged with a calcium carbonate deposit between the foundation and floor of the gallery. The pressure below the plug in some holes was sufficient to squirt to the roof of the gallery when the jet pipe broke through the plug. The source of the deposit has not been definitely established, but it is assumed to be coming from the concrete. The excessive uplift pressures in monolith 19 are attributed to inadequate number of drainage holes in an area of relatively impervious rock.

McNARY DAM

McNary Dam, located on the Columbia River, consists of concrete gravity spillway section, a combined intake and powerhouse section, and earth embankments tying into abutments at both ends. The hydraulic height is 90 ft. The foundation rock consists of basalt which ranges from closely jointed and fractured to massive structure. The basalt is underlain by an impervious sedimentary interbed of 40 ft to 60 ft in thickness. The dip of the strata is to the south, and the top of the interbed ranges from about 60 ft to 100 ft below the foundations for the spillway and powerhouse intake. The sedimentary interbed is underlain by additional basalt. Downstream from the dam deep potholes have penetrated the upper basalt providing an escape for reservoir water, as illustrated by Fig. 3. The grouting and drainage were performed from a drainage gallery near the foundation that extended through the powerhouse and spillway areas. In general, the grout holes were spaced on 5-ft centers and the drainage holes on 10-ft centers. The grout holes were drilled into the underlying impervious interbed to form an impervious cutoff. The drain holes were drilled about a third the depth of the grout curtain. In the powerhouse area, the gallery for grouting and drainage is located about 50 ft upstream from the intake gates as shown on Fig. 7. Uplift observations reported here are for the powerhouse intake. Uplift observations along the line of drainage holes were accomplished by plugging five adjacent drainage holes and observing the uplift pressure at each of these holes. The highest uplift always occurred at the center hole of the group plugged, so the uplift measurements do not represent uplift with all drains operating as designed. As shown by the section on Fig. 7, uplift observations were also obtained under the intake slabs downstream from the intake gates during the period that the units were unwatered. This was accomplished by observation in holes drilled through the intake slab. These



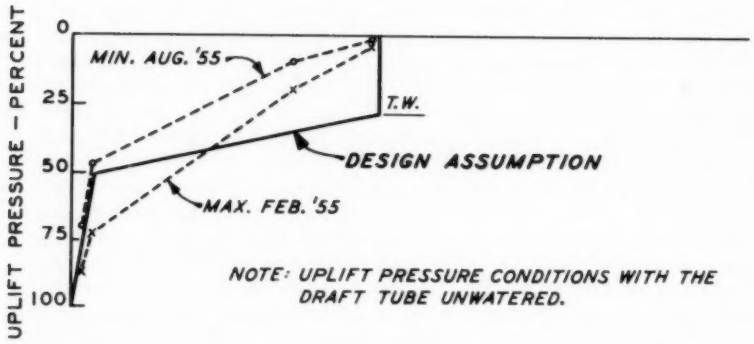


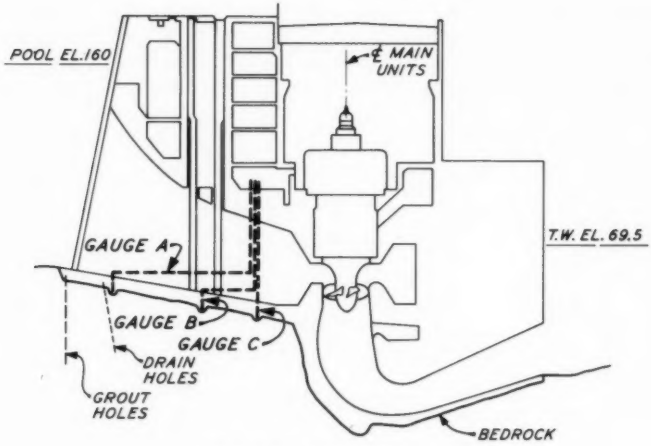
FIG. 7.—UPLIFT PRESSURES, McNARY DAM

holes were later replaced with essentially horizontal holes drilled from the gallery wall to the point for which observations were desired; the hole was plugged with a packer at this point and then grouted so that the pressures could be measured during operation of the power units. The pressures observed for these locations are less than the design assumptions with the units in operation. That the rock is not completely free draining is illustrated by the fact that the heads observed when the draft tubes were unwatered remained 20 ft to 30 ft above the pumped down water level. High uplift pressures were observed along the line of drain holes in monoliths 11, 12, 13, and 14 when the drain holes were plugged as described previously. In units 1 through 10 no excessive uplift pressures were observed. In reviewing the geological conditions in the foundations, it was noted that in units 1 through 10 the foundation rock is essentially highly fractured and jointed basalt, whereas in units 11 through 14 the foundation rock consists of massive dense basalt with a minimum of joints and fractures. The massive basalt with wide joint spacing does not provide an opportunity for drain holes, on 10-ft centers, to intercept many interconnecting joints and fractures; thus, the efficiency of the drainage is reduced. Closer drain hole spacing is indicated where the rock is not naturally free draining.

THE DALLES DAM

The Dalles Dam and powerhouse are located on the Columbia River and have a hydraulic height of 90 ft. The basalt foundation is similar to the basalts at McNary Dam just described, with the exception that there are no thick impervious interbeds. The individual basalt flows are vesicular in the upper portions and dense in the bottom two-thirds. The flow contacts and adjacent rock are the major pervious zones. A grout curtain with a maximum spacing of holes of 5 ft was constructed along the upstream edge of the intake slab. The average depth of grout holes was 100 ft. Drainage holes were drilled on about 14-ft centers to a depth of about 60 ft. The respective locations of the grout and drain holes are shown on Fig. 8. The openness of the foundation varied considerably as indicated by the flow of water into the draft tube foundation areas. The flow was only nominal for some areas but exceeded 500 g.p.m. in several units and was approximately 1500 gpm in one unit. The powerhouse and intake structure are a combined unit. Piezometers for measurement of uplift pressures were located in the foundation of the powerhouse intake structures. Fourteen of the 22 powerhouse units are completed with generating units, and units 15 through 22 are built as skeleton structures. The results of uplift measurements are shown in Fig. 8. It is evident from these observations that, generally, uplift pressures are considerably below the design values. The one observation that exceeded the design uplift was apparently caused by water trapped in a joint. Soon after the high uplift was noted in unit 1, a leakage of 13.5 gpm developed at the unit 1 - unit 2 contraction joint, accompanied by a drop of 24 ft in uplift head at this point. The initation of observations early on the first filling of the reservoir gives an opportunity to observe pressures for watered and unwatered units, unwatered units being those in which equipment is being installed and in which the draft tubes are unwatered and the intake gates closed. Uplift is influenced by tailwater to a varying degree. Watering of units 9, 10, and 11 did not change uplift pressures. However, watering of

unit 12 raised uplift heads on units 9, 10, and 12 to about tailwater. Units 8, 11, 20, and 21 indicate pressures below tailwater even though the draft tubes have been watered for several years. On the other hand, units 3, 6, and 7 suggest a very permeable foundation as evidenced by a sudden drop or rise in pressures after unwatering and watering of the draft tube of the same and even of an adjacent unit. Analysis of uplift pressure records leads to the conclusion



TYPICAL SECTION
INTAKE AND POWERHOUSE

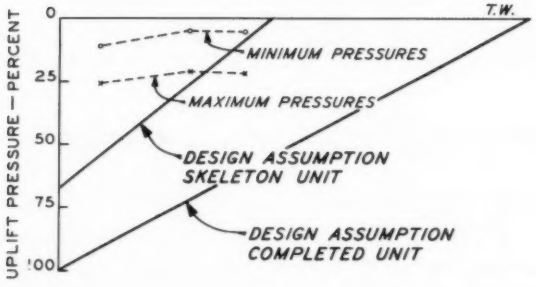


FIG. 8.—UPLIFT PRESSURES, THE DALLES DAM

that up to December, 1960, the uplift pressures in the foundation of the powerhouse intake are less than the design assumption. An attempt to correlate uplift conditions with geological conditions has been only partly successful. The openness of the foundation for unit 3 indicated by the fluctuation of uplift pressures with fluctuation of tailwater is confirmed by the grout take. About 3600

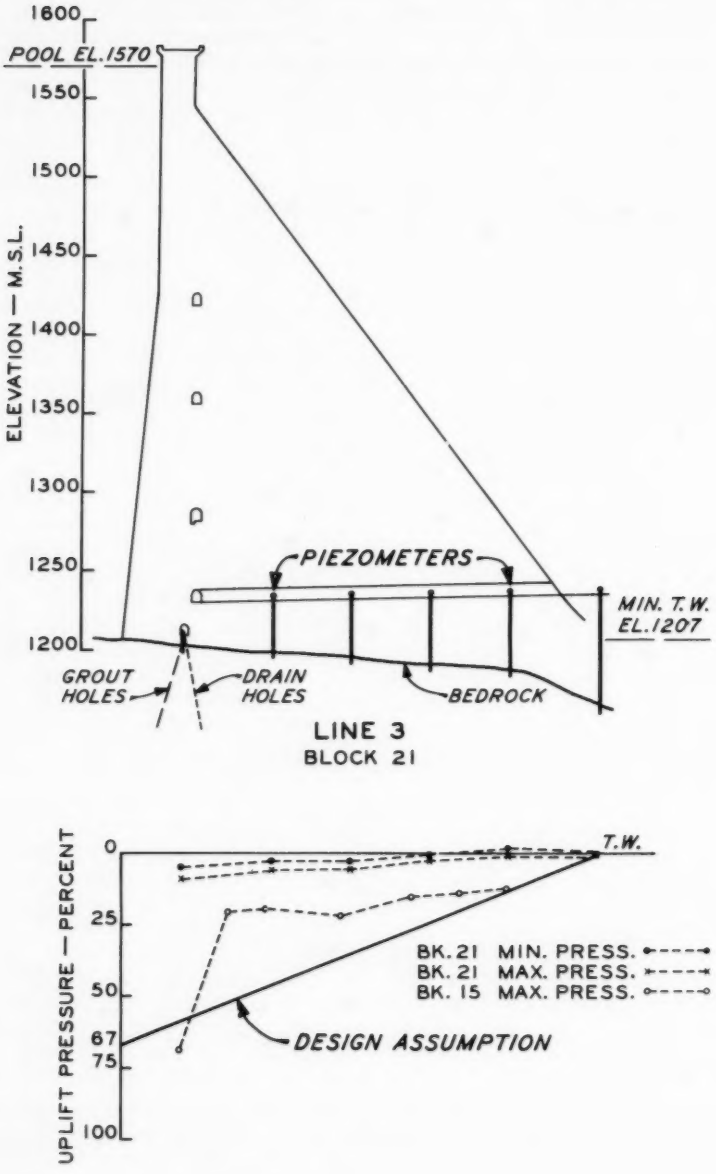


FIG. 9.—UPLIFT PRESSURES, DETROIT DAM

cu ft of grout was required in the foundation of this unit as compared with less than one-half that amount for most units.

DETROIT DAM

Detroit Dam, located on the North Santiam River in the Western Cascades, has a hydraulic height of 370 ft. The foundation consists of andesite breccia intruded by dikes of igneous rock. The rock in the river section is fresh and tight, whereas in the abutments it is weathered and more open. On the right abutment a steeply dipping clay-filled shear zone located between the grout curtain and the line of drainage holes supplements the grout curtain in reducing

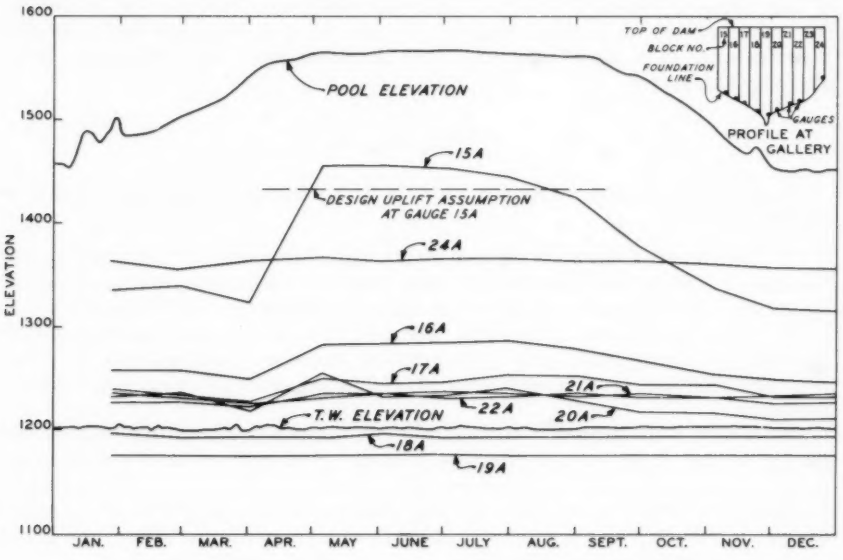


FIG. 10.—FOUNDATION UPLIFT PRESSURES, DETROIT DAM - 1959

leakage. The faults, veins, and fissures in a major portion of the foundation have a direction that is favorable to providing drainage downstream from the grout curtain. A massive andesite dike cuts diagonally across monolith 15 at about the midpoint. Foundation treatment consists of a grout curtain near the upstream face of the dam and a line of drainage holes inclined in a downstream direction located a few feet downstream from the grout curtain. The respective locations of these holes are shown in Fig. 9. The grout curtain consists of holes at a maximum spacing of 5 ft, to a maximum depth of 150 ft and is remarkedly tight as evidenced by the small amount of seepage from the drainage holes in the gallery. The total seepage into the gallery ranges from 22 gpm to 103 gpm,

depending on the reservoir stage. The drain holes are on 10-ft centers and are drilled to a maximum depth of 120 ft. Piezometers were installed in four lines normal to the axis of the dam, to observe uplift pressures at the foundation and in the foundation rock. Design assumption for uplift was full hydrostatic head at the upstream heel of the dam, decreasing to tailwater at the downstream face applied over two-thirds of the base area of the dam. As shown by Figs. 9 and 10, uplift pressures are, in general, considerably below the design assumptions. However, Gauge A, in monolith 15, showed a marked increase in pressure after the 1953 season. Because this high pressure continued, it was decided, in 1960, that action should be taken to reduce the pressure in this area. Five additional drainage holes have been drilled in monolith 15 with the result that the uplift pressure is 25% to 50% less than for previous years with the same reservoir level. In reviewing the geologic conditions, the presence of the massive andesite dike was the major difference between the foundation for monolith 15 and the foundation in the areas of the other three lines of piezometers. It is significant also that the foundation was so tight that little grout was accepted.

CONCLUSIONS

Based on the observations of the several concrete dams described it is concluded that:

1. Geological conditions have considerable influence on the uplift in the foundation and at the base of the dam. Detailed knowledge of the geological conditions facilitates the design of an adequate drainage and grouting system.
2. Where excessive uplift was observed it was because of an insufficient number of drainage holes in rock which had no interconnecting pattern of drainage. The addition of drainage holes reduced the pressures to well below design pressures.
3. Where well-defined pervious and impervious horizons are present the design of the grouting and drainage system is simplified and the cost minimized. Conversely where the nature of the rock is such that it is not possible to determine with any accuracy the path that water may take when it bypasses the grout curtain, the construction of the system is more costly because of the closely spaced holes that must be provided to assure adequate relief.
4. Most pressures observed were considerably below the design assumptions. The locations where excessive pressures were observed represent only a fractional percentage of the area at the base of the dam; however, it is possible that high pressures may occur at locations other than those for which piezometers are provided.
5. The provision of piezometers, to observe uplift at the foundation, has proved valuable for observing and controlling the uplift for existing structures and providing information for the basis of design for new structures. The use of drainage holes as a substitute for piezometers in determining uplift is unsatisfactory because it precludes measurements representative of natural drainage conditions.

ACKNOWLEDGMENTS

The cooperation of B. J. Bullert, Allen S. Cary, C. J. Monahan, L. L. Ruff, and F. W. Sims in providing geological information and uplift data for use in this paper is gratefully acknowledged.

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MULTIPLE SOIL SAMPLER

By P. J. Parsons[1]

SYNOPSIS

A portable sampling device has been developed that will collect several disturbed soil samples from various depths simultaneously. It is designed for use in sands and silts beneath the water table and is constructed from E drill rod. Normally twelve or more samples are collected at intervals of 2 ft, but the spacing may be varied by inserting lengths of drill rod between the sample units. The weight of the soil samples depends on the grain size and mobility of the material; typical samples are 60 g of fine sand or 30 g of silt. The device is not suitable for collecting samples of cohesive soils.

Normally two men are required to operate the sampler which may be driven with a light automatic hammer and extracted with a portable jack.

After repeated trials in soil that is contaminated at known horizons with radioactive fission products, it has been established by radiochemical assays that the samples are representative of the strata from which they were selected. The apparatus has been used regularly for eighteen months, and over 2,600 soil samples have been collected. In this program it has been usual to sample a borehole between 30-ft and 50 ft deep in one day's operation.

INTRODUCTION

At the Chalk River Project of Atomic Energy of Canada Limited, disposals of liquids containing radioactive fission products were made in the soil between

Note.—Discussion open until May 1, 1962. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Soil Mechanics Division, Proceedings of the American Society of Civil Engineers, Vol. 87, SM 6, December, 1961.

[1] Environmental Research Branch Atomic Energy of Canada Ltd., Chalk River, Ont., Canada.

1954 and 1955. It was known that these were migrating with the ground water away from their disposal sites, and it was important to discover the pattern and extent of this migration. If the front of the movement was to be located accurately, a detailed soil investigation was needed to sample soil at various depths from closely-spaced boreholes.

The terrain was composed of sand dunes bordered by swamp; water table contours had indicated that ground water was percolating beneath the sand hills towards the swamp.

A previous large scale soil-survey,[2] which had included the area to be examined, showed that sands and silts with small varved clay deposits were overlying glacial till.

DESIGN CONSIDERATIONS

Previous experience from sampling within a 4-in. casing had shown that the withdrawal of a casing produced a column of disturbed soil 6 ft to 8 ft in diameter having a different porosity from the neighboring material. This range of disturbance would be undesirable in a program containing boreholes that were closely spaced.

The apparatus to be used should be designed to cause only a small disturbance, be light enough to be carried through swamps, simple to operate, and be capable of collecting disturbed cohesionless soil from beneath the water table. To attain these conditions it was necessary to make the equipment as small as possible to be compatible with the size of the sample required.

DESCRIPTION OF APPARATUS

A light portable tool was developed that was easily driven, of simple structure, and efficient in being able to collect several samples of soil in one operation. It consists of a slender probe made from E size drill rod that is driven into the soil without a casing. Soil enters through slots cut at intervals in the side wall and passes into the sample chambers connected in series down the bore of the probe.

Fig. 1 shows a cross section of one sampler unit intermediate in a connected string of samplers. Each sampler unit is housed in a 2-ft length of drill rod; it comprises a double-piston unit used for controlling the input of soil and a connecting rod to the next sampler above it. The sample chamber lies between the upper and lower piston in the double piston unit. The apparatus is thus composed of a probe body of drill rods with their couplings and a core of double piston units joined together with connecting rods.

The drill rods are joined by couplings having a smaller bore, and the connecting rods between double piston units have been made small enough to allow them to pass through these couplings.

PROCEDURE FOR ASSEMBLY AND DRIVING

The probe is built up from a drive point screwed into the lowest drill rod; sections are added by fixing each double piston and connecting rod before

[2] "Movement of Radioactive Wastes Through Soil," Soil and Ground Water Investigations in Lower Perch Lake Basin, Chalk River Report CRER-932.

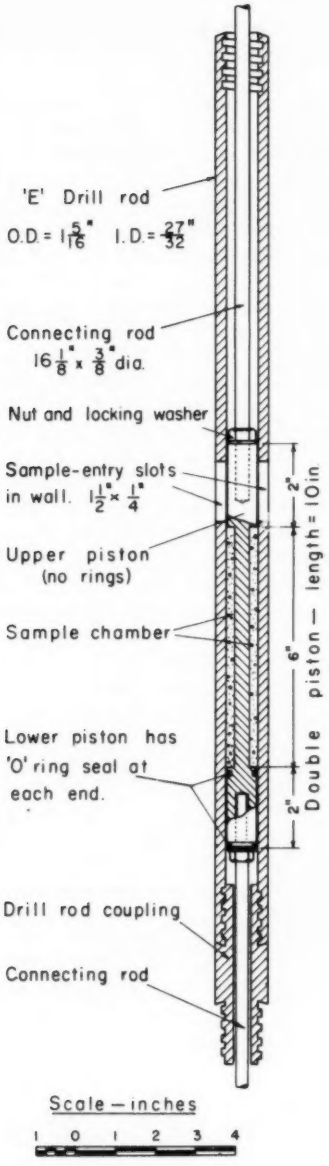

FIG. 1.—CROSS SECTION OF SAMPLER UNIT

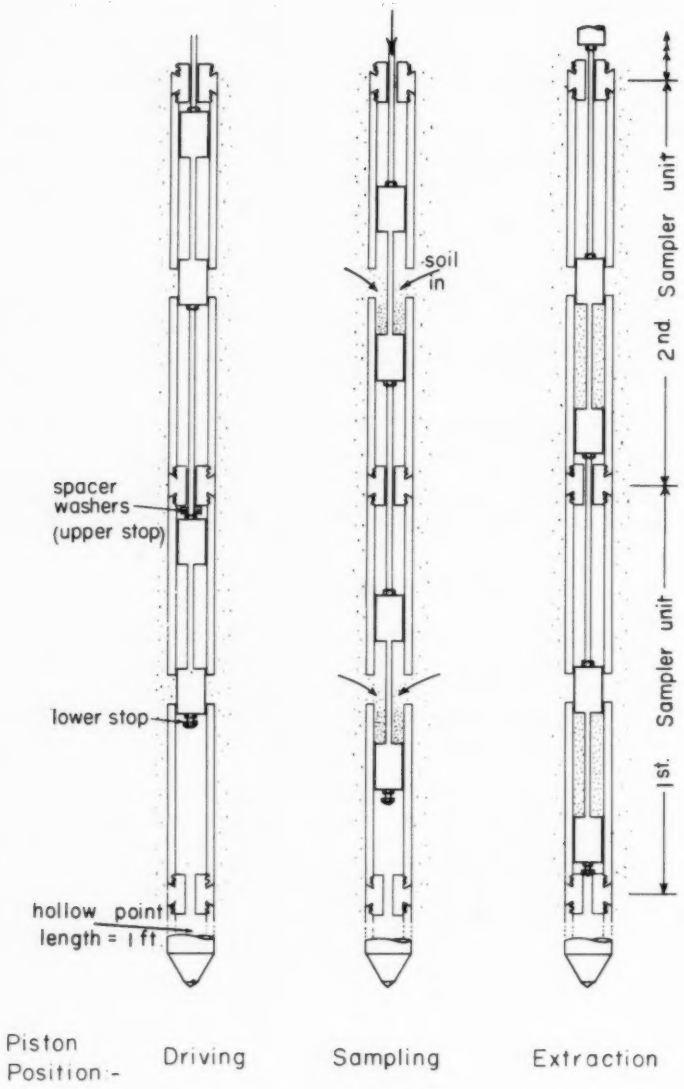

FIG. 2.—SAMPLING PROCEDURE

lowering the drill rod and coupling over it. After each sampler has been assembled, its connecting rod protrudes through the top coupling ready to receive the next piston unit. Normally four of these units have been pre-assembled while a starter hole for the sampler has been formed using a 1-1/2 in. ship's auger.

A diagrammatic illustration of the sampling procedure is shown in Fig. 2. Soil is excluded from the sampling chambers while the apparatus is being driven, by ensuring that the lower piston in each double piston unit is opposite the appropriate port in the side-wall. When the driving has progressed to the desired depth, the series of piston assemblies are forced down within the probe to allow soil to enter through each port. The sample is then sealed from the soil outside by forcing the pistons and samples down further until the upper pistons close the ports.

The probe is driven into the soil by a light automatic hammer that delivers its blows through driving accessories fixed to the upper drill rod. Fig. 3 shows these accessories in their assembled position. A length of drill rod (half a 2-ft section) is screwed on to the top coupling and is long enough to extend above the protruding connecting rod. A drive helmet, made from 2-in. diameter mild steel, is set on to the drill rod extension; it is bored at one end to receive the E rod and at the other end for the stubbed-end of a tool bit. This arrangement enables the hammer to stand unaided on the sampler and to be used at head-height without difficulty (Fig. 4). Although the piston units do not move in the bore under their own weight, it was found that the piston-to-wall friction was insufficient to restrain them against the hammer vibration in the drill rod. To ensure that the pistons are anchored during driving, a special coupling is used with the driving accessories; this has a set screw to clamp it to the connecting rod from the piston assemblies.

The samplers are driven until the hammer is almost at ground level, when additional samplers are connected and driven as required. Where ground water is several feet below grade, or soil samples are not required near the surface, the samplers may be driven further by connecting 5-ft lengths of drill rod and driving these as required. The piston assemblies are extended through these rods by installing a dummy piston (3 ft 10 in. long) in each rod and joining them together with standard connecting rods.

When the desired depth has been reached, the hammer is transferred from the drill rods to the top connecting rod protruding through the coupling. This is forced down between 2-in. and 6-in., thereby moving the pistons and exposing each sample chamber to the port and the surrounding soil. If the soil is sufficiently mobile it flows into the chambers with the ground water, but if not it may be induced to enter by vibrating the drill rods with the hammer for a few seconds. The piston assemblies are then further forced down until the total vertical displacement of the connecting rod is 8 in. At this stage the upper pistons have closed the ports, and the samples have been isolated from the outside.

WITHDRAWAL AND DISASSEMBLY

To withdraw the rods, a rack-jack is used in conjunction with a ball-cone clamp; Fig. 5 shows this apparatus. The jack functions from a hand-operated crank geared through a rack and pinion to a fork lift. The ball cone clamp slides smoothly in one direction over rods and couplings but is locked on to them through a conical ball race when forced in the other direction (Fig. 6).

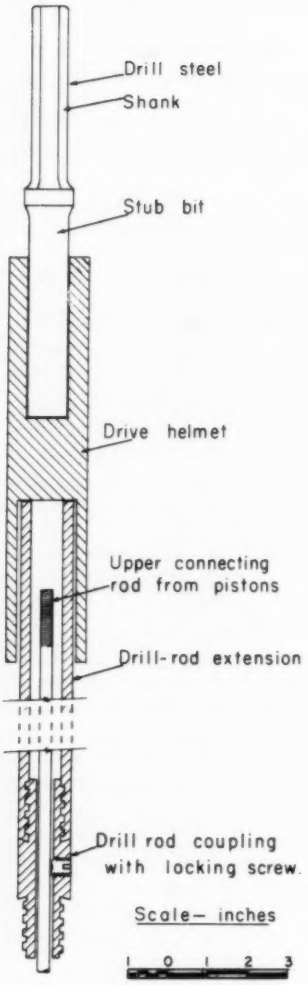

FIG. 3.—ACCESSORIES IN ASSEMBLED POSITION

The upper rods and dummy pistons are disconnected as they are jacked to the surface. To disassemble the samplers, each tube is raised until the top is about 3 ft above ground, the lock nut and washer are released from the protruding connecting rod, and the coupling and sampler body is unscrewed (1-1/2 turns). The loosened rod is withdrawn by hand to expose the piston

FIG. 4.—HAMMER AND SAMPLER ARRANGEMENT

FIG. 5.—RACK-JACK

with its column of soil, and a piece of sheet rubber, slit from the edge to its centre, is wrapped round below the piston and the sample scraped into this for removal (Fig. 7).

DETAILS OF SAMPLER

The drill rod forming the body of the probe is 1.313 in. outside diameter with a bore of 0.843 in.; mid-way along each 2-ft length are two milled slots

(1.5 in. by 0.25 in.). The soil enters through these diametrically opposed ports.

The double piston is formed from a 10 in. by 0.837 in. diameter stainless steel rod turned down to 0.437 in. diameter over the central six inches to leave 2-in. long pistons at each end. The ends are drilled and tapped 1 in. deep to receive the connecting rods, which are screwed in and secured with locking nuts and spring washers.

When the sampler was being developed, it was found that the vibration from the hammer caused fine particles of soil to enter through the ports and pass between the lower piston and the side-wall. Experiments with various piston clearances showed that 0.006 in. diametral clearance was satisfactory pro-

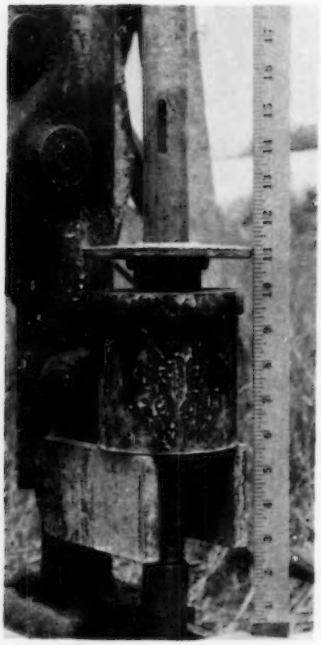

FIG. 6.—BALL CONE CLAMP

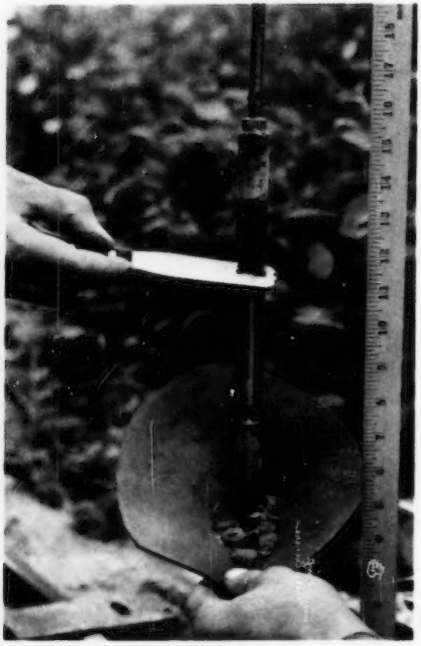

FIG. 7.—SAMPLE BEING REMOVED

vided that piston rings were fitted to the lower piston. Two rubber "O" rings (1820/12) were fitted in grooves (0.093 in. deep, 0.103 in. wide) at the ends of the lower piston, so that they were positioned immediately above and below the ports when the apparatus was being driven.

The upper piston is not fitted with sealing rings and the skirt clearance (0.003 in.) allows the slow escape of displaced air when soil enters the sample chamber. This piston masks the ports after the sample has been collected and is an adequate seal against the soil outside because there is no vibration when the sampler is being extracted.

Drill Rod Coupling.—It has been found that a seal is necessary at each coupling to prevent the entry of ground water past the semi-square threads. A liberal coating of water-pump grease over the threads of the couplings provides a satisfactory seal.

Piston Stops.—When the probe is being driven, the pistons are subjected to forces tending to move them upwards. It is therefore desirable to ensure that they are installed at the upper limit of travel when in the correct position to mask the ports. To achieve this, spacer washers are introduced above the double piston in the first sampler, their number providing the necessary adjustment (Fig. 2). Similarly, when the pistons are moved down until the upper pistons mask the ports, their position is adjusted to be the lower limit of travel by a stop screw fitted at the base of the lowest piston.

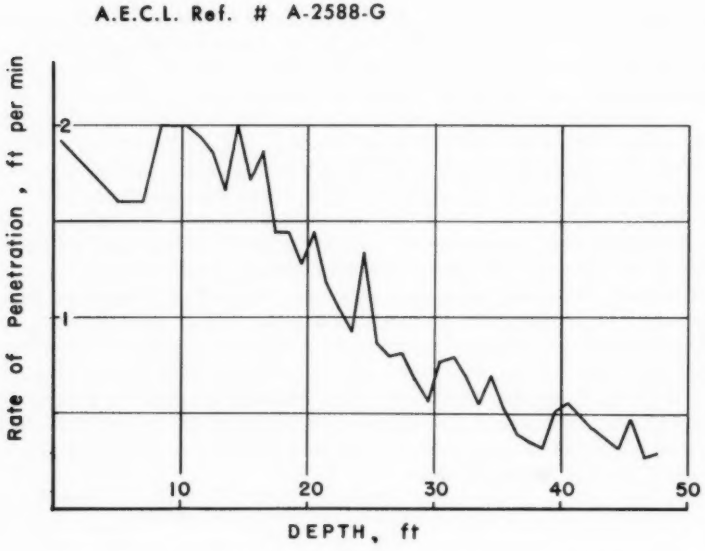

FIG. 8.—RATE OF PENETRATION ON A TYPICAL TEST

Drive Point.—A hollow drive point 1 ft long, screwed into the bottom coupling, is constructed from a short piece sawed from a length of drill rod with a driving shoe welded on to one end. This device is used to reduce the resistance from the compression of air trapped beneath the piston as the latter moves down during sampling.

OPERATING RANGE

The sampler is designed for use in cohesionless saturated fine sands and normally collects about 60 g in each sample. When used in coarser sands or fine grit, larger samples may be collected, but with silts the sample is often

reduced to 30 g. The device is not suitable for collecting clay samples. The maximum depth at which the sampler has been operated is 68 ft. This sample was the lowest one in a series of eight taken between depths of 54 ft and 68 ft.

In a test to see how many samples could be taken with one drive, twenty samples were collected; these were taken at 2-ft intervals between depths of 3 ft and 41 ft.

Rate of Sampling.—An Atlas "Cobra" hammer has been used throughout the sampling program at Chalk River. This weighs 53 lb (24 kg) and delivers 2300 blows per min with an impact energy of 13 ft-lb (1.8 kg m). The rate of penetration on a typical test is shown in Fig. 8 in which 2 ft per min was maintained to 15 ft, thereafter decreasing to 6 in. per min at 45 ft. During this test 48 ft was driven in 1 hr and 50 min, of which 1 hr was used in driving and 50 min in assembly. The extraction and disassembly of twelve samples lasted an additional hour.

Accuracy of Sampling.—During the experimental development of this tool, it seemed possible that soil could lodge in the thick-walled entry ports during driving and be carried down with the probe. Thus, when the ports were opened, the desired sample would be preceded by soil from an unknown level at lesser depth. In order to test this theory, the sampler has been driven repeatedly through layers of soil (16 ft thick) heavily contaminated with radioactive strontium. The samplers that penetrated into the deeper strata collected uncontaminated soil, and those samples from within the band were highly radioactive. These tests proved that the samples were selected from the desired horizons, but another experiment was required to determine whether these samples were truly representative of the strata at these levels.

Undisturbed soil samples were collected from neighboring positions, previously sampled by the probe, between the depths of 14 ft and 52 ft. By comparing the grading of corresponding samples, results indicated that the vibration of the probe caused some separation of the particle sizes. In the fine sands of this area 60% of the grains are usually between 0.1 mm - 0.2 mm diameter (sieves 140-70). The samples were approximately 9% low in this range and correspondingly higher in the proportion of coarser material. The silt content of the samples was about 2% high.

CONCLUSIONS

The sampler has been used by two men continually employed in its operation for eighteen months. During this period over 2,600 samples have been collected to produce a very detailed picture of the movement of radioactivity through the soil. It has proved a useful tool for preliminary soil investigation and has enabled boreholes for undisturbed sampling to be sited to the best advantage.

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STATIC AND DYNAMIC BEHAVIOR OF SMALL FOOTINGS

By E. T. Selig,[1] A.M. ASCE, and K. E. McKee,[2] M. ASCE

SYNOPSIS

A number of small footings resting on sand have been subjected to both static and dynamic loads. Load-displacement characteristics have been determined experimentally for a series of these footings subjected to static loading. The results of 82 experiments are correlated for square, circular, and rectangular footings. The load-displacement curves and typical sand failure patterns are included and examined, and the resulting bearing capacities are compared with the Terzaghi equations. The dynamic loads were created by the impact of a falling weight. The energy of impact is correlated with total footing settlement for 57 tests. The behavior under these types of static and dynamic load is compared with special emphasis given to the nature of the soil distortion. The difference in mode of failure of the sand for these two situations is clearly illustrated by photographs taken of both the surface of the soil and of a vertical plane through the soil.

INTRODUCTION

The subject of the behavior of footings under non-vibratory time-dependent loads presents an essentially virgin field for experimental investigation. Pre-

Note.—Discussion open until May 1, 1962. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. SM 6, December, 1961.

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vious studies have primarily considered response of footings to vibratory loads such as those induced by reciprocating machinery. The demands for foundations that are properly designed for other types of dynamic load, such as those designed to resist nuclear blast, have increased the need for further research effort. The experiments reported herein were undertaken to help fulfill this need.

Most experimental studies concerning the behavior of footings under static loads have concentrated their attention on the maximum bearing capacity of the footings with little or no attention given to the load-displacement characteristics. A demand for a better understanding of the behavior of footings under time-dependent loads has increased the need for information regarding these load-displacement relationships. In order to obtain this information, small footings placed on dense, dry Ottawa sand were subjected to static loading. The condition of the sand was maintained as nearly the same as possible from test to test; the footing shape, dimensions, and depth of burial were varied. The load-displacement characteristics so determined will be presented and analyzed.

Small footings placed on the same bed of sand were next subjected to impulsive loads in order to provide a comparison of the dynamic behavior with that of the same footings under static loads. For the dynamic tests the energy of impact is correlated with the total settlement of each footing. Using both two-dimensional and three-dimensional models of the footing-soil system, the soil distortions are then contrasted for the static and dynamic cases. The two-dimensional models provide a means of viewing sub-surface soil movements associated with footing failure.

EXPERIMENTAL CONDITIONS

The soil used in these tests was a uniform Ottawa sand (see Fig. 1 for gradation curve) kept air-dry throughout the tests. The specific gravity of the sand grains was 2.659. The minimum density that was obtained in the laboratory for this sand was 96.3 lb per cu ft. It was obtained by slowly pouring the sand into a Proctor mold from a constant height of approximately 1/2-in. above the surface. The maximum density that was obtained using a Proctor mold was 112.7 lb per cu ft. It was achieved by probing the filled mold with a 1/8-in. rod attached to a vibro-tool.

THREE-DIMENSIONAL EXPERIMENTS

The three-dimensional experiments were conducted in a box approximately 48 in. square and 36 in. deep. This box was built of wood and strapped with steel for added strength. Initially the box was filled with sand in 6 in. layers. The sand was vibrated by means of a 1-hp flexible-shaft concrete vibrator after each layer had been placed. Based on the known volume of the box and the measured weight of sand placed in it, the average sand density in the box after filling it initially was computed to be 112.3 lb per cu ft, a value nearly as large as the maximum density obtained in the Proctor mold. Prior to each test the sand in the box was vibrated for an additional five minutes while at the same time the surface was made smooth and flat. The initial average density was assumed to apply throughout the experimental program.

The footings were machined from aluminum plate in square, circular and rectangular shapes. They ranged in bearing area from 4 sq in. to 63 sq in. To simulate the roughness of an actual footing, the bottoms of the blocks were knurled. Surface footings were made 2 in. thick. Buried footings were built up to 2 in. above the surface, with additional aluminum blocks of the same horizontal dimensions as the footing.

The static loads were applied to the footing by a hydraulic jack acting through a calibrated loading ring as shown in Fig. 2. Dial gages supported on a cross member were used to measure deflections at three points on the top of the footing. The readings of the loading ring and the dial gages were recorded after each load increment. The footing displacements were taken to be the average of the deflections measured by the dial gages. Loads were obtained directly from the calibration charts of the loading rings. Following each test, photographs were taken of the failure pattern that formed on the surface of the sand, and sketches drawn to scale were prepared for each failure pattern.

The dynamic loads were applied by dropping a known weight onto the footing from specified heights. The falling weight was guided by a rigid slide which allowed essentially free fall, but which ensured that the flat bottom of the weight would strike the footing horizontally and at the proper point of contact. The impacting energy and total footing settlement were recorded and the failure pattern photographed for each test.

Two-Dimensional Experiments.—The two-dimensional experiments were conducted in a container with two glass sides 23 in. long by 17-1/2 in. high, spaced 4 in. apart (Fig. 3). The glass sides represent vertical planes through a bed of soil. The footings used in the glass box, being made to extend the full 4-in. width of the container, simulate a unit length of an infinitely long footing. It has been found that the friction between the sand and the glass sides is small enough so that the tests provide good qualitative two-dimensional representation.

Prior to each test with dense sand the glass box was filled with sand and vibrated for 3 min by means of a 1-hp flexible-shaft concrete vibrator (the same as used in the sand box) clamped to its top. This technique resulted in an average sand density of 109.0 lb per cu ft which was found to be applicable for all of the tests on dense sand. Possible density variations within the box were not determined. For the tests with loose sand the glass box was filled by slowly pouring the sand from a constant height. The average sand density in this case was 98.5 lb per cu ft. A grid (1/2 in. spaces and 1/16 in. thick lines) made of black sand was placed on the sand surface adjacent to one of the glass sides so that the movement of the sand could be readily observed and photographed.

The footings were machined from aluminum plates 2 in. thick and the bottoms were diamond knurled as for the three-dimensional tests. In these two-dimensional tests all of the footings were 4 in. long to fit between the glass plates and either 2 in. or 3 in. wide. The method of loading for both the static and dynamic tests was the same as in the three-dimensional case.

THREE-DIMENSIONAL STATIC RESULTS

The results of the three-dimensional static experiments are summarized in Table 1. The data for all of the types of footings studied is presented along

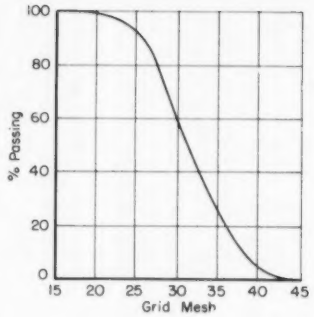

FIG. 1.—GRADATION CURVE OF UNIFORM OTTAWA SAND USED IN EXPERIMENTS

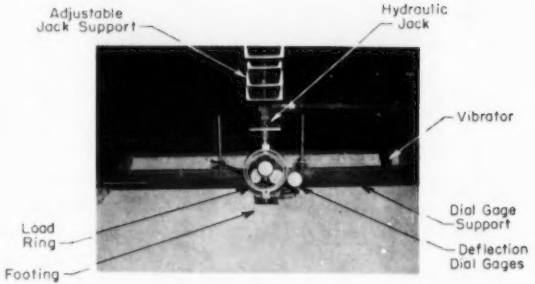

FIG. 2.—GENERAL EXPERIMENTAL SETUP

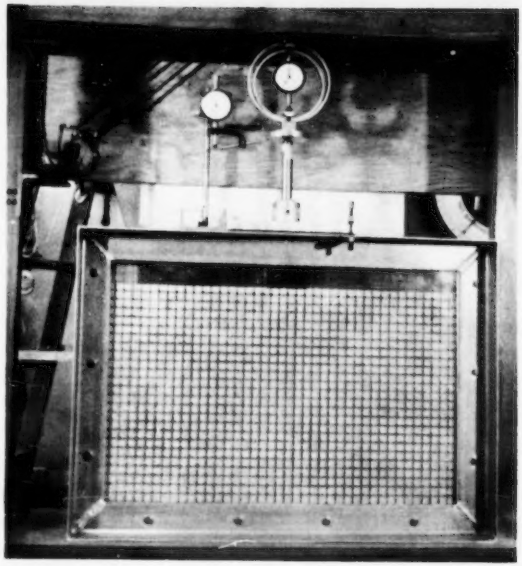

FIG. 3.—GLASS BOX APPARATUS

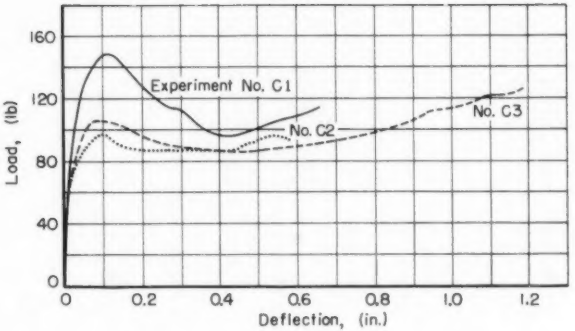

FIG. 4.—LOAD-DISPLACEMENT MEASUREMENTS FOR 3-IN. SQUARE FOOTINGS

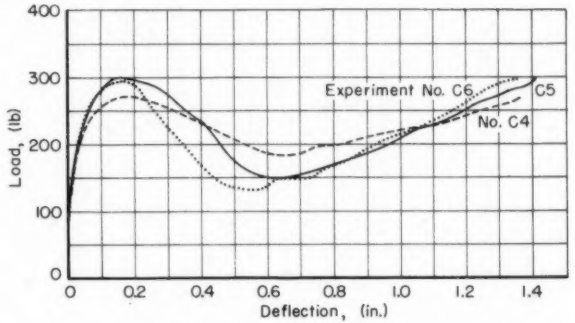

FIG. 5.—LOAD-DISPLACEMENT MEASUREMENTS FOR 4-IN. SQUARE FOOTINGS

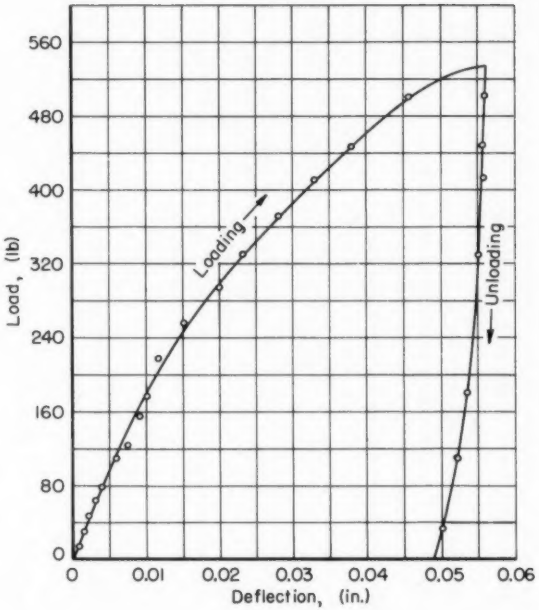

FIG. 6.—LOAD-DISPLACEMENT CURVE FOR 3-IN. BY 9-IN. FOOTING SHOWING EFFECTS OF UNLOADING

with the average values for the maximum load and bearing capacity. Of primary interest in the present study are the load-displacement characteristics. As an example of the results obtained, three load-displacement curves for a 3-in. square surface footing are shown in Fig. 4, and three similar curves are shown for a 4-in. square surface footing in Fig. 5. the behavior of footings at relatively large deflections is typified by these curves. Fig. 4 demonstrates that the reproducibility of the loads associated with large deflections may actually exceed the reproducibility of the initial peak loads. Both Figs. 4 and 5 show essentially the same shape for footings on dense sand, that is, an initial

TABLE 1.—RESULTS OF THREE-DIMENSIONAL STATIC TESTS

Footing Shape	Dimensions, inches	Area, Square inches	Depth of Burial, inches	No. of Footings Included	Avg. Max. Load, pounds	Range of Bearing Capacities, pounds per square inch	Avg. Bearing Capacity, pounds per square inch
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Square	2 x 2	4	0	5	37	6.75–10.0	9.3
Circular	2.26 diameter	4	0	7	37	5.25–9.8	9.3
Square	3 x 3	9	0	9	136	10.8–18.9	15.1
			2	5	391	37.8–54.5	43.5
			4	4	711	70.4–87.5	78.9
			6	4	964	94.9–119.3	107.0
Circular	3.39 diameter	9	0	5	127	13.5–14.7	14.1
Square	4 x 4	16	0	11	305	15.1–21.8	19.1
Rectangular	3 x 6	18	0	6	305	12.7–17.6	15.9
	3 x 9	27	0	6	472	16.2–19.4	17.5
	3 x 12	36	0	5	812	19.7–26.8	22.5
	3 x 15	45	0	3	1015	21.9–23.8	22.5
	3 x 18	54	0	4	1175	19.7–24.5	21.7
	3 x 21	63	0	3	1453	22.4–24.0	23.1

peak followed by a valley and a gradual rise to values equalling or exceeding those of the initial peak. The general behavior of footings at large displacements is of major importance when one wishes to consider the failure of footings under dynamic loads. In the remainder of this paper attention will deal primarily with the maximum bearing capacities because they are of more general interest.

Fig. 6 shows the deflection rebound on unloading a 3-in. by 9-in. rectangular surface footing just prior to application of the maximum load. The unloading occurs along a curve with a slope much greater than the slope of the load-

ing portion, resulting in a large hysteresis as expected. The recoverable displacement is approximately 0.007 in. compared with the total displacement of 0.056 in. For displacements beyond those associated with the maximum load, the recovery on unloading is equally small and can be assumed to be essentially zero when compared with the total displacements.

Figs. 4 and 5 are intended to give an indication of the general form of the load-displacement curves and the reproducibility which can be achieved from supposedly identical experiments. The range of bearing capacities (Table 1) also gives an indication of the reproducibility. Later in this paper the effects of the various footing parameters will be evaluated by comparisons of the bearing capacities. Photographs and sketches of failure patterns that were observed on the surface of the sand were made for each test. Figs. 7 through 9 are typical of these observations. In general, the intersections of the failure planes with the surface of the sand were clearly visible. At the moment when the bearing capacities of the footings were reached, differential displacements of the sand surface began to indicate the failure zone. With increased displacements of the footing beyond this point it was evident from the surface displacements that, in general, the location of the failure surface did not change. However, in approximately 10% of the footing tests, a second failure surface was observed to form (as in Fig. 8a) immediately after the first. In no instance did this shift result in an abrupt change in the load-displacement curve.

Theoretically, two distinct types of failure are possible: (1) Symmetrical failure, and (2) one-sided failure. The load was applied in two ways (indicated in Table 1): (1) Directly through a plate rigidly attached to the base of the loading ring so that the footing would be prevented from rotating, and 2) through a ball located at the center of the footing so that the footing would be free to rotate. Within the accuracy of the experimental setup used for this research, there was no observable preference for symmetrical or one-sided failure with either method of loading (this would obviously not be true for eccentric or inclined loads) and the load-displacement curves, including the peak values were essentially the same for either type of failure.

Figs. 7, 9, and 15 show symmetrical failure patterns. Figs. 7a and 7b are typical of those obtained for circular footings, and Fig. 15 for square footings, whereas Fig. 9 is for rectangular footings. The only significant distinction between the patterns for the circular and square footing shapes is the irregularity of the failure pattern around the square footing which does not occur around the circular one. In other tests on square footings one-sided failure patterns were produced. Such a pattern is shown in Fig. 8. The inner pattern is formed first, followed by a switch to the outer pattern. Other tests on the rectangular footings also produced one-sided failure patterns. Although the failure patterns have been referred to as one-sided or symmetrical, most of the patterns were actually intermediate to these two idealized types.

TWO-DIMENSIONAL STATIC RESULTS

Differential movement of the sand surface during failure of footings for the three-dimensional static tests have indicated the existence of a clearly defined failure zone through the soil. The purpose of the two-dimensional static tests was to provide more information on the exact nature of this failure zone.

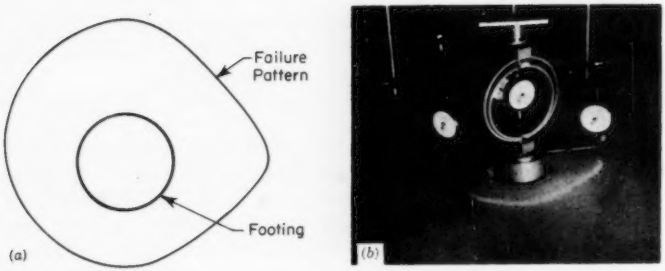

FIG. 7.—SYMMETRICAL FAILURE PATTERN FOR 9 SQ. IN. CIRCULAR SURFACE FOOTING

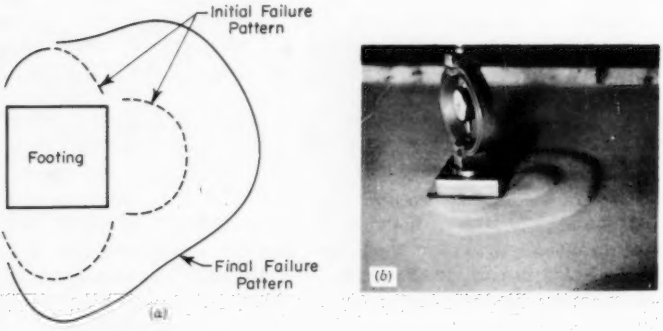

FIG. 8.—PHOTOGRAPH OF DOUBLE FAILURE PATTERN FOR 4-IN. SQUARE SURFACE FOOTING

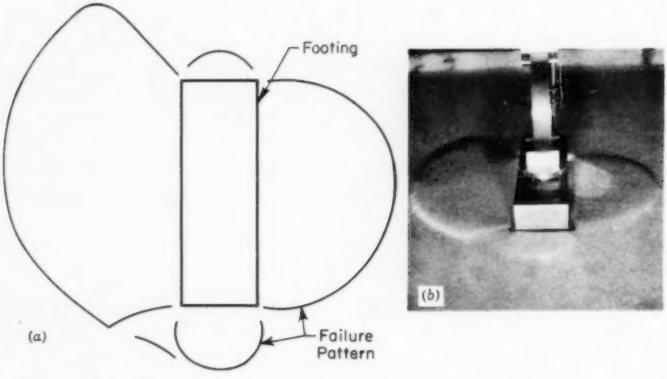

FIG. 9.—NEAR-SYMMETRICAL FAILURE PATTERN FOR 3-IN. BY 9-IN. RECTANGULAR SURFACE FOOTING

The sand distortion below the surface for static tests in the dense sand (density = 112.3 per cu ft) may be clearly seen from the following results of these two-dimensional static tests. Fig. 10 shows a symmetrical failure quite similar to that of the Terzaghi modified Prandtl theory.[3] Because these footings, although loaded symmetrically, were not constrained against rotation, a nonsymmetrical pattern such as shown in Fig. 11 frequently occurred. The latter figure is an example of a completely one-sided failure of a buried footing. In all these examples there are clearly defined shear surfaces extending outward from the footing to the sand surface.

In contrast to the general shear type of failure under static load, another type, the local shear failure,[3] occurs when there are no well-defined shear planes. Instead the distortion of the sand extends continuously outward from the bottom of the footing as shown in Fig. 12. It should be noted that local shear failure may, upon continued loading be eventually followed by shear surface formation. The example of the local shear type of failure was obtained by placing the sand in the glass box in a loose state (density = 98.5 lb per cu ft).

THREE-DIMENSIONAL DYNAMIC RESULTS

The three-dimensional dynamic tests are illustrated in Figs. 13 and 14. In Fig. 13 prior to the test, the sand has been vibrated, the surface leveled and a grid of colored sand laid out to help in determining the movement of the surface during the experiment. The guide for the falling weight may be seen extending downward from the top-center of the picture above a 3-in. square footing. The bed of sand and the footing with the weight still resting on it are shown in Fig. 14 after the weight had been dropped.

It may be readily observed that a difference in the surface displacement of the sand occurs if the footing is loaded dynamically than if it is loaded statically with the soil conditions the same. This difference may be most easily seen by comparing Fig. 15 showing the surface after static loading of a 4-in. square footing with Fig. 16 showing the surface after dynamic loading of the same footing. In contrast to the static pattern, the dynamic pattern does not exhibit a single clearly defined shear surface. Part of the mound of sand surrounding the footing after the impulsive loading was created by throwout of sand from around the edge of the footing while the rest was sand pushed up as the footing settled under the impact.

Weights were dropped on 57 footings placed on the surface of the sand box with identically prepared dense sand beds (density = 112.3 lb per cu ft). The total settlement of these footings due to the dynamic load together with the height of the drop were recorded. The results of these tests for square footings are shown in Fig. 17 where the footing settlement associated with the dynamic load is plotted versus the drop height. These experiments indicated a linear variation of settlement versus drop height for the soil conditions used.

Because no clearly defined shear surfaces were observed when the impulsive load was applied to these footings, it was decided that for purposes of comparison, dynamic failure would be considered to have taken place when the footing settlement had reached or exceeded the settlement at failure for the same footing loaded statically. The static settlements associated with the maximum loads were equal to or less than 0.16. In all except two of the dynamic

[3] "Theoretical Soil Mechanics," by K. Terzaghi, John Wiley and Sons, Inc., New York, 1943.

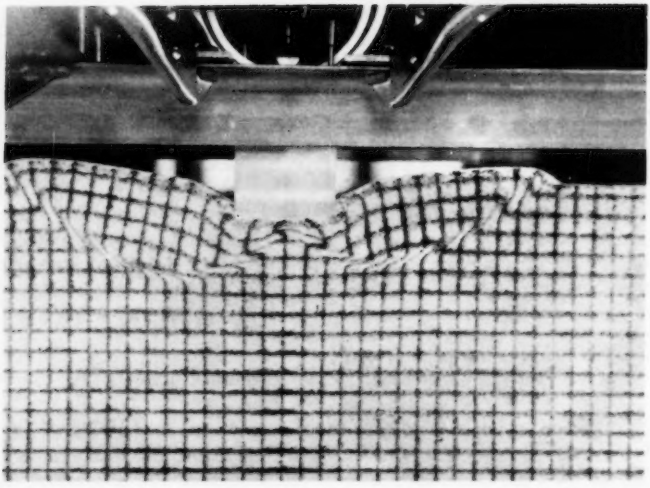

FIG. 10.—STATIC TEST OF 3-IN. FOOTING AFTER FAILURE

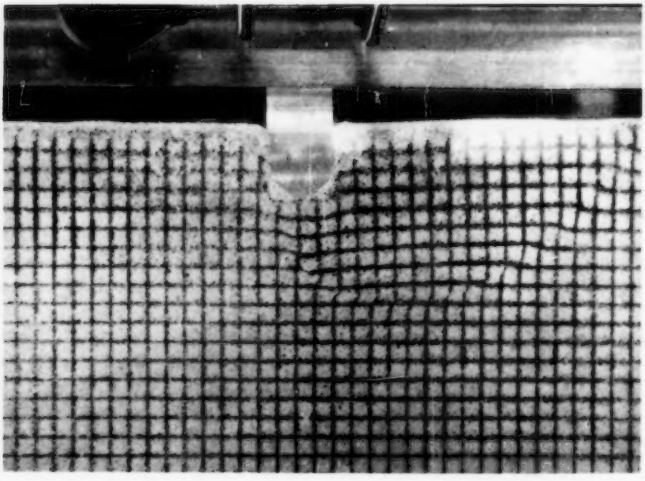

FIG. 11.—STATIC TEST OF 2-IN. FOOTING BURIED 2 IN. AFTER FAILURE

tests (D54 and D55) the total settlement was greater than the static failure settlement. Hence, using this criterion of failure the characteristics of soil distortion shown for the dynamic tests are those associated with footing failure under impulsive loads.

TWO-DIMENSIONAL DYNAMIC RESULTS

The nature of the sand distortion below the surface due to dynamic loads may be seen from the results of the two-dimensional dynamic tests with the same dense sand. Fig. 18 shows the distortion beneath a 3-in. wide footing obtained by dropping a 29 lb weight 20 in. The resulting total settlement was 0.469 in. It may be noted that although this type of distortion is quite different

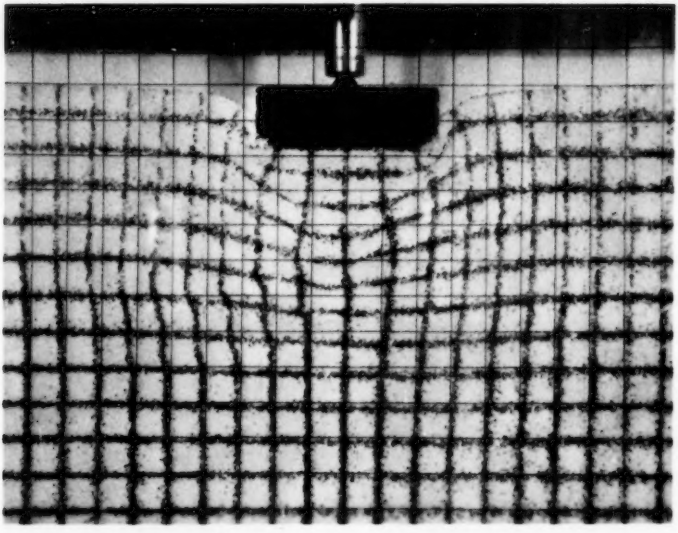

FIG. 12.—LOOSE SAND STATIC TEST IN GLASS BOX

from that for the dense bed of sand with the footing loaded statically, that is, the general shear failure pattern, it is similar to that for the loose bed of sand with the footing loaded statically (Fig. 12), that is, local shear failure pattern.

Fig. 19 shows the distortion after two additional impacts of the same intensity on the footing shown in Fig. 18. The total settlement for the footing after the third drop was 2.156 in. It is of interest to note that under the third impact in the latter case a shear plane has occurred superimposing itself on the previous dynamic pattern. Within the limitations of this experimental study such

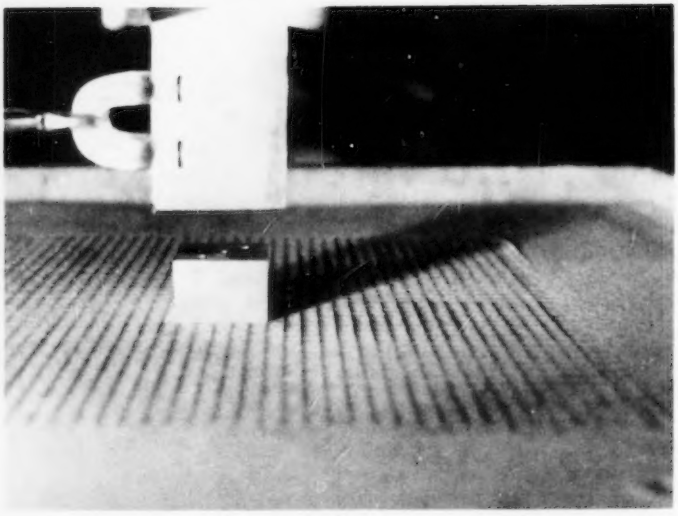

FIG. 13.—3-IN. FOOTING IN PLACE FOR TEST

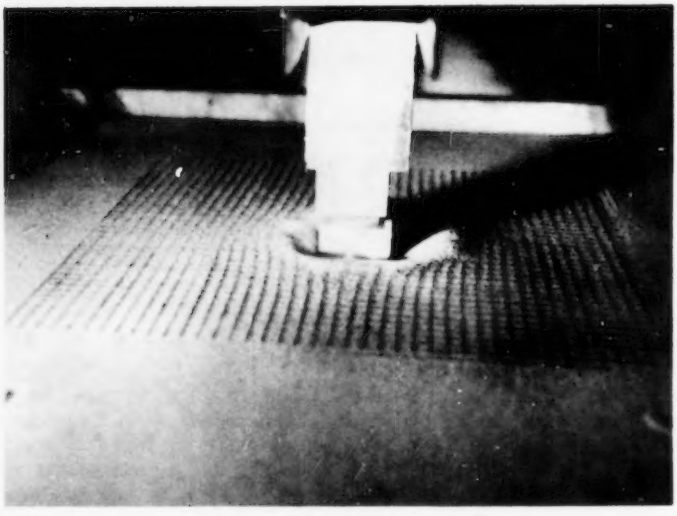

FIG. 14.—3-IN. FOOTING TEST AFTER DROP (DROP WEIGHT IN PLACE)

shear failure occurred only on drops subsequent to the first and, hence, are likely to be the result of the changed boundary conditions due to the first drop.

BEARING CAPACITIES COMPARED WITH THEORY

The experimentally determined bearing capacities taken as the first maximum pressure from the static load-deflection curves can be compared with

FIG. 15.—SURFACE AFTER STATIC TEST 4-IN. SQUARE FOOTING

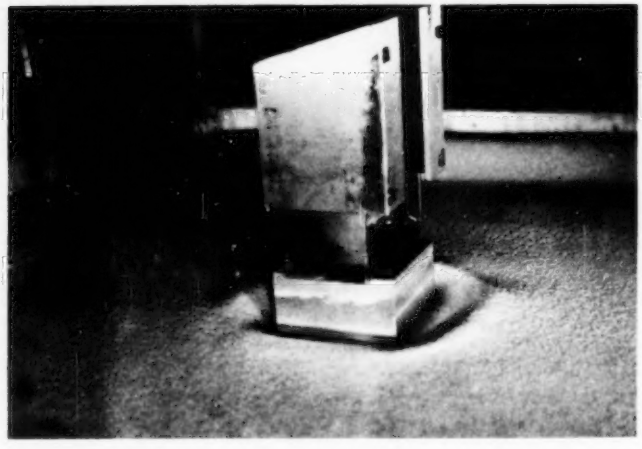

FIG. 16.—SURFACE AFTER DYNAMIC TEST 4-IN. SQUARE FOOTING

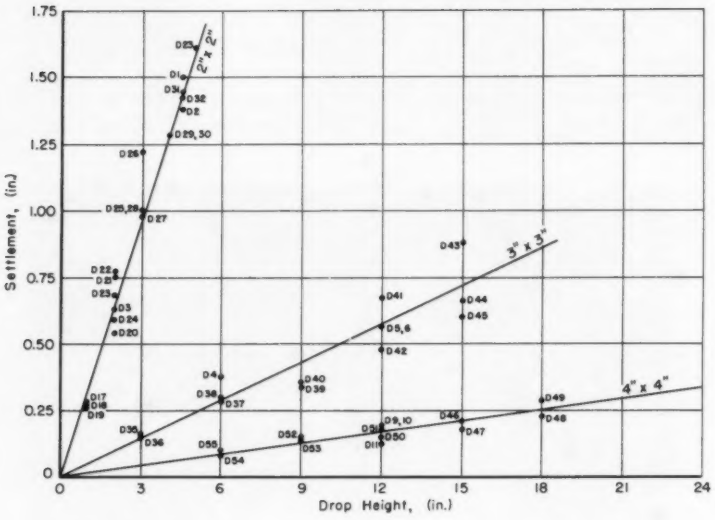

FIG. 17.—DROP HEIGHT VERSUS SETTLEMENT FOR SQUARE FOOTINGS (DROP WEIGHT = 11.2 LB)

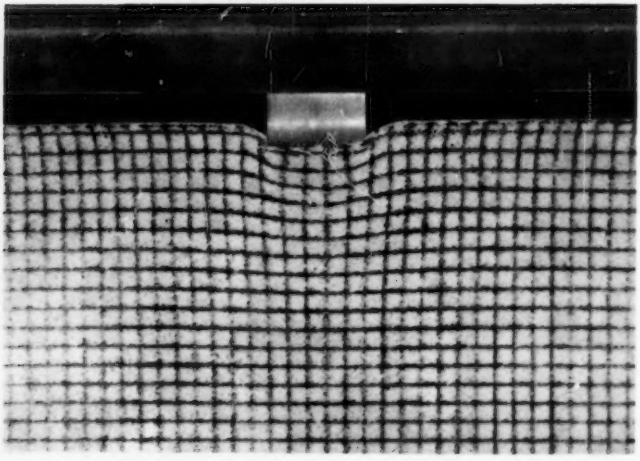

FIG. 18.—DYNAMIC TEST OF 3-IN. FOOTING

available theoretical formulas. Such comparisons can be made in many ways—the extent of the comparisons being limited only by the time devoted to the study. For the purpose of this examination, all comparisons will be made with Terzaghi's approximate formulas for the bearing capacity of footings.[4] These formulas are perhaps the best known, and have been the most frequently used for comparison with other available data and theories.

The bearing capacity formula developed by Karl Terzaghi, Hon. M. ASCE, for a continuous footing is

$$q = c\,N_c + \gamma D N_q + \frac{1}{2}\,\gamma\,B N_\gamma \quad \ldots\ldots\ldots \quad (1)$$

in which N_c, N_q and N_γ are dimensionless bearing capacity factors dependent only on ϕ; B denotes the footing width, in inches; c is the cohesion, in pounds

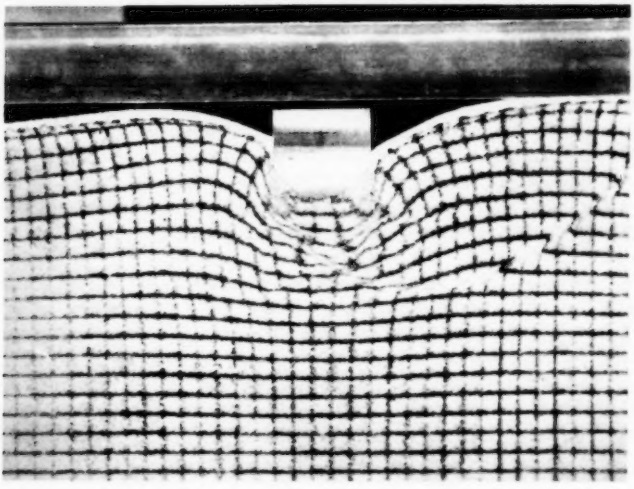

FIG. 19.—DYNAMIC TEST OF 3-IN. FOOTING, THIRD DROP

per square inch; D describes the depth of burial, in inches; q is the bearing capacity, in pounds per square inch; γ is the density of soil, in pounds per cubic inch; and ϕ denotes the angle of internal friction of the soil, in degrees. If consideration is limited to surface footings (D = 0) on dry sand (c = 0), Eq. 1 reduces to

$$q = \frac{1}{2}\,\gamma\,B N_\gamma \quad \ldots\ldots\ldots\ldots \quad (2)$$

Terzaghi's empirical modifications of these formulas for square and circular footings are

$$q_\phi = 0.6\,\gamma\,r N_\gamma \quad \ldots\ldots\ldots\ldots \quad (3)$$

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4 "Soil Mechanics in Engineering Practice," by K. Terzaghi and R. B. Peck, John Wiley and Sons, Inc., New York, 1948.

and

$$q_{\square} = 0.4 \gamma B N_{\gamma} \quad \ldots\ldots\ldots\ldots \quad (4)$$

in which q_{ϕ} denotes the bearing capacity for circular footings, in pounds per square inch; $q_{\square}$ is the bearing capacity for square footings, in pounds per square inch; and r refers to the radius of circular footings, in inches. For all of these formulas it may be noted that the bearing capacity is directly proportional to the dimensions of the footing.

By using the data from Table 1, Figs. 20 and 21 were prepared. Both the average and range of experimental values are shown. A comparison with Eqs. 3 and 4 can be made for square and circular footings using these figures. In both cases, a linear variation plotted through the origin and the experimental points look extremely reasonable, thus confirming the linearity of these equations. Because circular and square footings of equal area were tested, it is possible to compare the theoretical and experimental bearing capacities of footings having these shapes. For an equal area, the ratio of Eq. 3 to Eq. 4 gives the ratio of bearing capacity of a circular footing to that of a square footing to be 0.845. The experimental results give 1.000 for this ratio and 0.887 for the 4-sq. in. and 9-sq. in. footings, respectively.

Fig. 22 shows the effect of length on the bearing capacity of 3-in. wide footings. Based on Terzaghi's formulas (Eqs. 2 and 4), the ratio between the bearing capacity for a square and an infinitely long footing would be 0.80. Using the experimental results for the 3-in. by 3-in. and the 3-in. by 21-in. footings, the ratio is 0.69.

It is interesting to compare the predictions (Eq. 1) for effect of depth of burial with the corresponding data obtained in the laboratory, although these data probably involve depths which are out of the range in which the equation is normally applicable. The series of experiments with the 3-in. by 3-in. footing at various depths of burial are plotted on Fig. 23. It may be seen that the result is a straight line with a slope of 18.2 psi per in. If Eq. 1 applies, this slope should equal γN_q, which for γ = 112.3 lb per cu ft gives N_q = 2.80. This value of N_q is approximately twice that associated with the ϕ found for surface footings.

More quantitative comparisons can also be made between the experimental data and Terzaghi's formulas. By means of triaxial shear tests, values of ϕ ranging from 38° to 41° were predicted for the Ottawa sand used in the experiments at densities of 112 to 113 lb per cu ft. Using this ϕ in Terzaghi's equations gives bearing capacities smaller than those actually observed in the laboratory. Similar discrepancies have been observed by other investigators when dense dry sand at low stress levels is studied.[5]

For the purpose of this analysis the comparison of theory and experiment will be made by using the experimentally obtained bearing capacities to determine the angle of internal friction which results in agreement between the laboratory results and the equations. The bearing capacity values given in Table1 were inserted into Eqs. 2, 3, and 4 to determine the required ϕ. The results are tabulated in Table 2 for the surface footings. The quantity N_{γ} was

5 "The Ultimate Bearing Pressure of Rectangular Footings," by H. Q. Golder, Journal, I. C. E., Paper No. 5274, London, England, 1942.

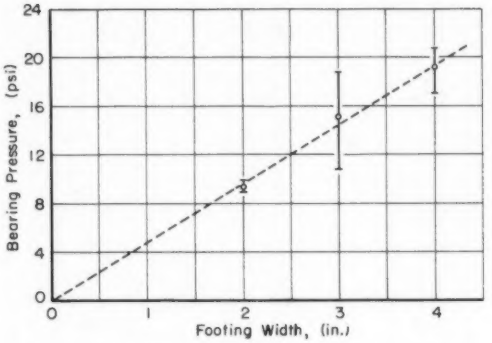

FIG. 20.—EFFECT OF FOOTING DIMENSIONS FOR SQUARE FOOTINGS

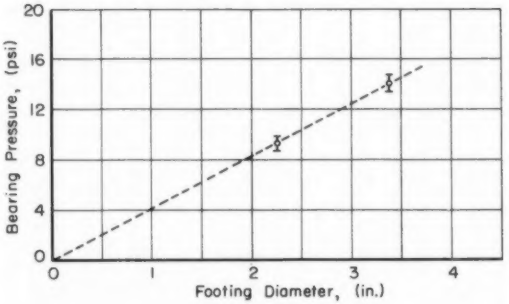

FIG. 21.—EFFECT OF FOOTING DIMENSIONS FOR CIRCULAR FOOTINGS

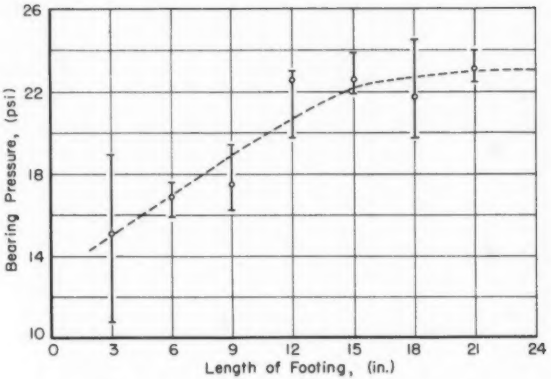

FIG. 22.—EFFECT OF FOOTING LENGTH

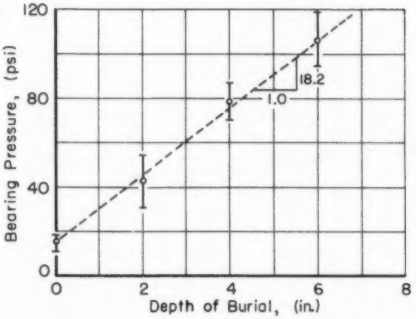

FIG. 23.—EFFECT OF DEPTH OF BURIAL

computed and converted to ϕ using the values given by Terzaghi and Ralph B. Peck F. ASCE.[6]

The computed values of ϕ based on the average experimental results with dense sand are extremely consistent (varying from 41.3 to 43.3). This uniformity of results occurs with the use of Terzaghi's empirical coefficients for

TABLE 2.—SOIL PROPERTIES BASED ON MEASURED BEARING CAPACITY

Footing Shape	Dimensions, inches	Avg. Max. Bearing Capacity, pounds per square inch	N_γ	ϕ degrees
(1)	(2)	(3)	(4)	
Square	2 x 2	9.3	179	41.8
Circular	2.26 diameter	9.3	211	42.8
Square	3 x 3	15.1	204	42.5
Circular	3.39 diameter	14.1	213	42.8
Square	4 x 4	19.1	187	42.0
Rectangular	3 x 6	15.9	163	41.3
	3 x 9	17.5	180	41.8
	3 x 12	22.5	231	43.3
	3 x 15	22.5	231	43.3
	3 x 18	21.7	222	43.1
	3 x 21	23.1	237	43.3
	Averages =		205.3	42.55

N_γ = dimensionless bearing capacity factor (refers to soil density)

ϕ = angle of internal friction of the soil, degrees

square and circular footings and treating the rectangular footings as if they were infinitely long.

SUMMARY AND CONCLUSIONS

A number of small footings resting on dense sand have been subjected to both static and dynamic loads. One of the major purposes of these tests was to determine whether or not the same mode of failure exists in these two situations. For the static tests in dense sand the classical general shear failure occurred. However, for equal or greater total settlement under the dynamic load an entirely different mode of sand distortion occurred. The mode was

[6] "Soil Mechanics in Engineering Practice," by K. Terzaghi and R. B. Peck, John Wiley and Sons, Inc., New York, 1948, p. 171.

very similar to that of the local shear failure that occurs for the same footings loaded statically in loose sand. Only after the third repeated impact on the same footing was there any evidence of formation of a well defined shear plane for the dynamic tests. And in these cases the total settlement was quite large (about 75% of the footing width).

The shape of the static load-settlement curves have been shown and comparisons made for variations in depth of burial, length of rectangular footings, and size of square and circular footings. The bearing capacities taken as the first maximum of these curves were then compared with the Terzaghi equations. The linearity of the relationship between the bearing capacity and the footing dimensions was confirmed for square and circular surface footings. For an equal footing area, the ratio of bearing capacity of a circular footing to that of a square footing was found to be within 20% of the same ratio from Terzaghi's empirical equations. The ratio of bearing capacity of square footings to that for an equal width footing of large length-to-width ratio was found to be 14% less than the Terzaghi value. The increase in bearing capacity with depth was approximately twice the value predicted by the Terzaghi equation. Using a value of $\phi = 42.3° \pm 1°$ all of the bearing capacities of the surface footings were in agreement with the Terzaghi values. This value of ϕ is in reasonable agreement with measured values of ϕ from triaxial tests at high densities.

The total settlement for the dynamic tests was found to increase linearly with energy of impact for each individual footing. The settlement for a given impact energy became larger at an increasingly greater rate as the size of the footing decreased.

The two types of loading used, static and impulsive, represent two opposite extremes. For these two cases a different mode of failure was observed for dry sand. It is to be expected that there will be a transition from one mode to the other at some intermediate rates of loading. The behavior may also be quite different for different soil types. Many additional studies will be required to understand fully and to be able to predict the behavior of footings under dynamic loading. However, it is hoped that the preliminary studies reported herein have served to demonstrate the type of behavior that might be expected in practice.

ACKNOWLEDGMENTS

The experimental investigations discussed in this paper were carried out under the sponsorship of the Research Directorate, Air Force Special Weapons Center, (AFSWC) Air Research and Development Command, Kirtland Air Force Base, N. Mex.[7] Particular acknowledgment is made to H. Mason A. M. ASCE, C. Wiehle, M. ASCE, of AFSWC who had direct cognizance of the technical work and who made many valuable suggestions that were incorporated into the results presented herein. The writers also wish to acknowledge the assistance of J. Mulay, A.M. ASCE, in conducting the laboratory work, and of E. Vey, F. ASCE, Supervisor of Soil Mechanics and Professor of Civil Engineering at the Illinois Institute of Technology, in serving as a consultant for the investigation.

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[7] "Design and Analysis of Foundations for Protective Structures," by K. E. McKee, AFSWC-TR-59-56, October 15, 1959.

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DISCUSSION

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EXPERIENCE WITH A PIER-SUPPORTED BUILDING OVER PERMAFROST[a]

Closure by H. B. Dickens

H. B. DICKENS.[6]—Edward F. Lobacz's report of the apparently satisfactory performance of a foundation at Bethel, Alaska, similar in design to the building described by the writers, is of much interest, as are his comments on the research findings of the Arctic Construction and Frost Effects Laboratory in Fairbanks, Alaska. These experiences further substantiate the suitability of this type of foundation as an alternative to embedded foundations in permafrost areas and at the same time serve to emphasize the care with which items such as drainage control must be handled during construction to achieve success.

One significant difference between the design described by Lobacz and that described by the writers should be noted. This relates to the removal, in the examples cited, of considerably more subsurface material prior to placing the non-frost susceptible fill. In the case of the building at Bethel, as much as 11 ft of such material was removed. This additional excavation tends to negate the advantages of this system of construction and may make the avoidance of thermal disturbance more difficult. Details were not given, but presumably this depth of excavation was considered necessary because of local permafrost and soil conditions, Bethel being in an area at or close to the southern boundary of permafrost in Alaska. Churchill, the site of the building described by the writers, is within the zone of continuous permafrost in Canada, making it possible to found the concrete pier foundations directly on the permafrost at a depth of only $2\frac{1}{2}$ ft below the original ground surface.

As Lobacz implies, it is unfortunate in the case of the Churchill building that more information correlating such factors as soil temperatures and change in level of the permafrost was not obtained. As the paper explained, the building was originally not the subject of a research study. However, such studies of existing buildings should be encouraged, as they do provide an economical and useful method of obtaining much valuable information on the performance of structures in permafrost areas.

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[a] October 1960, by H. B. Dickens and C. M. Gray (Proc. Paper 2618).
[6]Div. of Bldg. Research, Nat'l. Research Council, Ottawa, Canada.

GROUTING OF GRANULAR MATERIALS[a]

Discussion by Emile Huni and John E. Wagner

EMILE HUNI.[30]—The paper aroused keen interest, because the writer has been connected with the design and construction of the deepest grouting project in granular materials completed to date (1961).

Some of these projects, in Europe as well as in Canada, in which the grouting method, designated by the authors as the "cuffed tube" method, has been extensively used will be described. This grouting method is more widely known as the "sleeved grout-pipe" method, and it has recently been applied in the United States (for the first time) in the construction of the grout curtain under the "slurry trench" at Wanapum Dam on the Columbia River, Washington.

The authors' description of the method is accurate and needs no further explanation. However, it should be mentioned that the "weak cement-sand grout" that fills the annular space between the grout-pipe and the grout-hole consists of a weak cement-clay grout that retains a certain plasticity after setting. A cement-sand grout would become brittle, and the plasticity of this "sleeve grout," as it is often called, is a prime requisite to successful grouting through sleeved grout-pipes.

The first extensive grouting using the "sleeved grout-pipe method" was at Bou-Hanifia Dam in Algeria in 1935-1937. The foundation of the dam consisted of lenses of marly sandstone with beds of marly sand and conglomerates overlaying an impervious marl. The granular materials were grouted using a silica-based gel with an inert charge of coarse clay. The deepest groutholes were approximately 300 ft deep, and 44,500 ft of sleeved grout-pipes were used. The total length of groutholes amounted to some 204,000 ft.

However, Bou-Hanifia Dam cannot be considered a typical application of the sleeved grout-pipe method, because the grouted formations generally had a non-negligible cohesion.

In 1942-1945, a 5-row grout curtain, extending over approximately 27,000 sq ft, was constructed at the Lac Noir Dam in the Vosges Mountains, France. The foundation consisted of washed-out morainic loose materials, that caused significant leakage under the existing dam, with an appreciable amount of washed-out fine sands, that endangered the stability of the structure. Some 9,200 ft of sleeved grout-pipes were used for grouting cement-clay mixes, as well as silica-based gels with an inert clay charge. Sand lenses were grouted using a pure silica-based gel. The maximum depth of the groutholes was approximately 85 ft. They were rotary drilled, using shot and mud. Huge boulders, several feet in dimension, were encountered.

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[a] April 1961, by John C. King and Edward G. W. Bush (Proc. Paper 2791).
[30] Cons. Engr., Vancouver, B. C.

In 1952-1956, the 400-ft deep grouted cutoff of Serre-Poncon Dam was constructed in the loose alluvials of the Durance River, in Southern France. Exceptionally thick at the top (12 rows of groutholes), in order to ensure a perfect bond with the vertical clay core of the 400-ft high earthfill dam, the grout curtain decreases in thickness with depth; only three rows of groutholes remain in the narrow deep furrow of the alluvial valley. The grout curtain extends over approximately 50,000 sq ft. Clay and slag cement grouts were used, with a higher cement/clay ratio in the external rows; they were pumped through approximately 25,000 ft of sleeved grout pipes placed in churn-drilled groutholes, using drilling mud.

Another 400-ft deep grouted cutoff was built for the Sylvenstein Dam, in 1955-1957, in the alluvial filled valley of the Isar River, Southern Germany. It consists of seven rows of groutholes at the top and two rows only at the bottom. The other characteristics of this grout curtain are similar to those at Serre-Poncon.

In 1958-1959, the 500-ft deep grouted cutoff for Mission Dam was constructed in the alluvial filled valley of the Bridge River, British Columbia. This grout curtain consists of five continuous rows of groutholes from the top to the bottom, totalling approximately 48,000 ft of rotary, mud-drilled groutholes. Because of special local conditions, only 28,000 ft of groutholes were equipped with sleeved grout-pipes, extending over a grouted area of some 68,000 sq ft. The remaining 20,000 ft, that were not grouted, were equipped with plain pipes. On the external rows, cement-clay mixes were used for grouting. The intermediate rows were grouted with more intruding clay-cement grouts having a higher clay/cement ratio. Finally, the central row was grouted with a grout made of chemically defloculated and rigidified clay.

Under construction is a 200,000 sq ft, 400-ft deep, grouted cutoff at Mattmark Dam, in Switzerland. Sleeved grout-pipes are used exclusively for clay-cement grouting.

All these structures are permanent cut-offs for dams, but extensive use of the sleeved grout-pipe method has also been made in temporary structures, such as cutoffs under cofferdams. The method has also been used in the protection of excavations in which "horizontal grout curtains" have been constructed, extending over areas up to 120,000 sq ft, at a depth of 150 ft below natural soil level. Frequent and successful use of this grouting method has also been made in the consolidation of soils for tunneling purposes.

A particular application of the sleeved grout-pipe method is to be found at Dartford Tunnel, in England. It is assumed that the grouting works at Dartford Tunnel described by the authors are those that were carried out during construction of the 10-ft diameter pilot tunnel in 1936. The sleeved grout-pipe method, on the other hand, was applied on grouting works done in 1956 during construction of the definitive 30.5 ft diameter highway tunnel. The excavation, using compressed air, had to be driven through a highly pervious alluvial layer of sand and gravel, located at a depth of 50 ft to 60 ft between the Thames clay and the chalk, on the north bank as well as on the south bank of the river Thames. During construction of the pilot tunnel through this layer of alluvials, heavy air losses were observed that, extrapolated to the diameter of the definitive tunnel, would have made the project uneconomical. The layer of alluvials has been sealed off against air losses by grouting cement-clay and deflocculated clay grouts, using sleeved grout-pipes, around the tunnel on its northern and southern sections, that passed through sands and gravels. The result was that

the air losses during construction of the main tunnel were much smaller than those observed on the pilot tunnel.

The advantages of the sleeved grout-pipe method have been briefly described by the authors. The method has also proved itself in shallow grouting, under critical conditions. If adequate care is given to the design of the sleeve grout, the seal between grout-pipe and grout-hole, extending to soil surface, permits accurate and adapted grouting of the formations.

Furthermore, after years of studies and development, it has been possible to perfect the sleeved grout-pipe method, whose original purpose was to effectively pump adapted grout into predetermined areas selected on the basis of soil investigation, to the point at which the amount of grout can now be accurately forecast. This has been done on the more recent large-scale projects with complete success. It has been possible to estimate grout quantities within a narrow margin; however, such accurate estimates are feasible only when subsoil conditions have been thoroughly investigated.

Acknowledgments.—The sleeved grout-pipe method has been developed and applied over the past 25 yr by Soletanche of Paris, France, and the companies of the Rodio Group, to whom the writer extends his appreciation for all the data that he has been able to compile during nearly 20 yr experience with them.

JOHN E. WAGNER,[31] F. ASCE.—It is the opinion of the writer that grouting of fine-grained soils with suspension-type grouts (or even chemical solution-type grouts), as described in the section entitled "Objectives," cannot be expected to produce acceptable results. Fracturing of a deposit to form sheets and bulbs of grout is wasteful and may actually increase the over-all permeability and reduce the stability of the deposit.

In the analysis of the data obtained during the grouting of the cutoff at Rocky Reach Hydroelectric Power Project,[32] the writer attempted to use the information presented by the authors concerning the influence of pore size on grouting materials.[33] For the sake of clarity in reference to the groutability ratios presented by the authors, the writer took the liberty of designating the ratio D_{10} formation to D_{95} grout as N', in order to distinguish it from the ratio D_{15} formation to D_{85} grout designated as N in this and other papers.[34,35,36]

In the explorations and planning for the Rocky Reach Hydroelectric Power Project, many borings and churn drill holes were made through the general area of the east abutment. From these borings, many spoon samples of the deep deposits of sands and gravels were obtained. The information from these bor-

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31 Capt., Corps of Engrs., Dept. of the Army, Mil. Asst. to the Director, U. S. Army Engr. Waterways Experiment Sta., Vicksburg, Miss.

32 "Symposium on Grouting: Construction of Rocky Reach Grouted Cutoff," by W. F. Swiger, Proceedings, ASCE, Vol. 87, No. SM2, April 1961.

33 "Construction and Performance of the Grouted Cutoff, Rocky Reach Hydroelectric Power Project," by J. E. Wagner, thesis presented to the University of Illinois, at Urbana, Illinois, in 1961, in partial fulfillment of the requirements for the degree of Doctor of Philosophy.

34 "Cement and Clay Grouting of Foundations: The Use of Clay in Pressure Grouting," by G. A. Kravetz, Proceedings, ASCE, Vol. 84, No. SM1, February, 1958.

35 "Cement and Clay Grouting of Foundations: Grouting with Clay-Cement Grouts," by S. J. Johnson, Proceedings, ASCE, Vol. 84, No. SM1, February, 1958.

36 "Grouting of Foundation Sands and Gravels," T. M. No. 3-408, U. S. Army Engr. Waterways Experiment Sta., Vicksburg, Miss., June, 1955.

ings and from well pumping tests made in the east abutment area indicated that beneath the east abutment terrace were several buried river channels filled with extremely permeable sands and gravels. These deposits had to be cut off or reduced in permeability in order to make feasible the construction of a dam at the Rocky Reach site. A grouted cutoff was selected for this purpose.

Although the general character of the buried channels was known, only a limited number of soil specimens were obtained from which sieve analyses could be determined. The writer attempted to use these limited data to relate the groutability ratio concept to the successful grouting results obtained at Rocky Reach. None of the soil specimens, mostly sandy gravels, for which sieve analyses were made came from the exact location of the grout curtain, and many of the soil particles in these specimens had been fractured as a result of the sampling operations. Nevertheless, these data were compared with the data available concerning the grout properties. The first of the two suspension-type grouts used at Rocky Reach was a clay-cement grout having D_{95} and D_{85} sizes of approximately 0.119 mm and 0.032 mm, respectively. More than 99% of the particles in the second grout, a cement-bentonite grout, were finer than 0.74 mm. Comparisons of the soil and grout grain size data predict that on the basis of groutability ratios, only 10% of the zones sampled could be penetrated with the clay-cement grout, and perhaps 20% could be penetrated with the cement-bentonite grout. Although some difficulty was encountered in penetrating the sand and gravel strata in the curtain area using the clay-cement grout, results were not so bad as the groutability ratios would have predicted. Use of the cement-bentonite grout was also considerably more successful than the groutability ratios would have predicted on the basis of the limited number of soil specimens available. However, by working backward from the minimum acceptable groutability ratio ($N' = 8$), the writer estimated that the bentonite-cement grout could have penetrated sands with a D_{10} size somewhat smaller than 0.59 mm, a medium to coarse sand. The results of the grouting operations appear to verify this conclusion.

Although the limited data available seem to verify the validity of the groutability ratio concept, its general applicability has many limitations. Undisturbed samples from deep cohesionless deposits are expensive and difficult to obtain. Any disturbance to the soil specimens, such as fracturing of the particles or mixing of the specimen, will result in misleading conclusions as to the groutability of a deposit. There may be a considerable difference in the groutability of a deposit, depending on whether it consists of alternating thin lenses of sands and gravels or it is a uniform mixture of sands and gravels. It is not possible to determine which condition may exist if the sampling methods have disturbed the sample. Whereas most of soil specimens obtained at Rocky Reach were mixtures of sands and gravels, geologic indications and exposed cuts suggest that most of the sands and gravels exist there as alternating lenses, rather than as sand-gravel mixtures.

In conclusion, the writer would like to point out that, in their discussion of pore diameters and densities of sands, the authors have used the term void ratio for porosity or per cent voids. The horizontal scale on Fig. 1 should read "porosity," not "void ratio."

SYMPOSIUM ON GROUTING: RESEARCH IN FOUNDATION GROUTING WITH CEMENT[a]

Discussion by Robert E. White

ROBERT E. WHITE,[5] F. ASCE.—With reference to the sections dealing with "Grout for Stabilizing Soft Soils ..... for Mixed-in-Place Piling ..... for Mixed-in-Place Cut-off Wall," the writer infers from Thomas B. Kennedy's text, including citation of references, that the method described may be relied on to yield satisfactory results. The writer's first-hand experience with intrusion mortar mixed-in-place piling leads him to the opposite conclusion.

In the belief that much may be learned by unsuccessful experiences, the following is related:

A few years ago, a vertical press was to be installed in a large New England metal working plant. A deep pit with concrete walls and floor was required—it was to be 50 ft by 14 ft in plan and 46 ft deep below the plant floor. Soil conditions were 0 to 8 ft depth—miscellaneous Fill; 8 ft to 13 ft depths—Fine Sand with Silt and Mica; 13 ft to 19 ft depth—Medium Fine Sand with Mica; 19 ft to 36 ft depth—Gravel with Stones, Boulders and Binder; 36 ft to 51 ft depth—Hardpan with large stones. Ground water level was 8 ft below the plant floor.

Bids were invited for the construction of the pit: the writer's firm submitted a bid of $196,000, the plan being based on (a) a dewatering method of using a well made by driving a large open end steel pipe to below subgrade, installing a gravel packed screen and a deep well pump, and then pulling up the steel pipe and (b) sheeting and bracing by means of soldier beams driven at 5 ft or 6 ft on centers, horizontal louvre board sheeting (packed with salt hay, if necessary), steel wales, and braces. A short time after the bids were received, the writer's firm was informed that another contractor was awarded the work.

Approximately six months later, the writer's firm was called back to complete the work. It was learned that the contractor to whom the work was initially awarded had elected to use intrusion mortar mixed-in-place piling using the methods described by Kennedy. These methods were principally developed by this same contractor, who was generally regarded as having the most experience in this type of work. The mixed-in-place piles were supposed to be about 14 in. in diameter, spaced at approximately 12 in. on center, and penetrate to below the subgrade, that as stated above, was 46 ft deep, so as to form a continuous cut-off wall inside of which bracing could be placed as the excavation was carried down. When subgrade was reached, the reinforced concrete slab was to be poured and the walls installed. The tops of the mixed-in-place piles, above ground water level, were to be encased in and braced by a heavy collar

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[a] April 1961, by Thomas B. Kennedy (Proc. Paper 2794).

[5] Exec. Vice Pres., Spencer, White & Prentis, Inc., New York, N. Y.

or wale of reinforced concrete, that was of such large size that no cross braces were required.

Unfortunately, the actual performance fell short of the plan, and when the original contract was cancelled, matters stood as follows: First, the intrusion mortar mixed-in-place piling, instead of penetrating to more than 46 ft below the plant floor, in no case went to a depth greater than 26 ft. Second, approximately one out of every five or six piles was non-existent, or practically so, because a number of piles were no more than 3 in. to 5 in. in diameter, whereas others simply were not there at all—the mixed-in-place grout evidently had in some cases been pulled up on the auger while it was being extracted or else diluted by ground water boiling up as the auger was being withdrawn; in any event, a far-from-continuous "cut-off" wall had been formed. Third, the piling

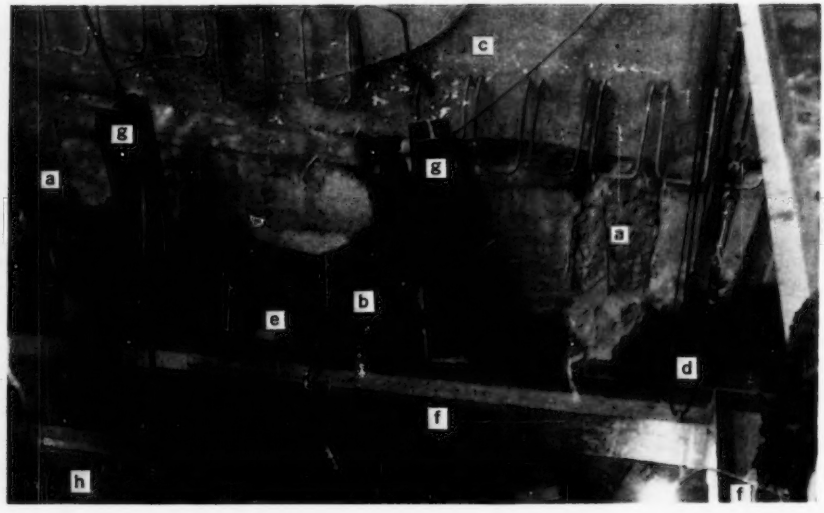

FIG. 1.—PRESS PIT CONSTRUCTION STARTED BY INTRUSION MORTAR MIXED-IN-PLACE PILING AND COMPLETED BY SOLDIER BEAMS AND HORIZONTAL LOUVRE BOARDS

was not in a straight line, and many piles had to be chipped back by as much as one-half their diameters to make room for the future concrete walls of the pit. Fourth, the excavation within the pit had been carried down to a depth of 16 ft, at which elevation the ground was quick with numerous boils in the bottom; this caused a great deal of concern, as heavily loaded combined roof and crane columns on spread footings close by were felt to be endangered, although fortunately no appreciable settlement had yet taken place. Fifth, the large reinforced concrete collar had been poured from El. -8 to -13; removing it presented a formidable problem, and it created a difficult constriction in the working space. Sixth, a large quantity of chemical grouting (sodium silicate and calcium chloride) had been injected into the earth at the spaces where the

mixed-in-place piling had failed to materialize, but this chemical grouting had not succeeded in any way in solidifying the ground; traces of the grout were found as lenses of a white deposit of cream cheese consistency. Seventh, the owner had paid approximately $75,000 to bring the work to this stage.

The pit was completed by means of dewatering wells, 8 in. H-beams, and horizontal sheeting, as originally planned by the writer's firm. Because of the constricted room, the louvre boards had to be blocked out in back of the soldier beams by means of steel sections, and the soldier beams were incorporated in the walls of the pit. Three tiers of 12 in. BP 53 lb wales and braces were used to support the soldier piles. The cost of completing the work was $168,000.

Fig. 1 shows the press pit construction started by intrusion mortar mixed-in-place piling and completed by soldier beams and horizontal louvre boards.

Point	Comment
a, a.	Intrusion mortar mixed-in-place piling. Note that both of these are off line and have had to be cut back to provide clearance. Pile at left is only about 8 in. in diameter; one at right is full size.
b.	Intrusion mortar mixed-in-place pile only about 5 in. in diameter.
c.	Reinforced concrete collar at top of pit encasing tops of intrusion mortar mixed-in-place piles.
d.	Horizontal louvre boards spanning between Intrusion Mortar Piles where latter were entirely missing.
e.	4 in. x 4 in. wooden strut to brace long span of horizontal louvre boards.
f, f.	12 in. 53 lb wales.
g, g.	8 in. BP 36 lb soldier beams.
h.	Horizontal louvre boards between soldier piles.

The writer's conclusion is that the method of constructing a continuous cut-off wall by means of overlapping intrusion mortar piling mixed-in-place by augers is unreliable. This conclusion is borne out not only by the case previously but also by personal observation of a more recent job in New York City, in which a cut-off wall was constructed by the same method by the same experienced intrusion mortar mixed-in-place piling contractor. This wall was to protect buildings from the effects of a deep adjacent sewer excavation, and it served this purpose. However, the wall was not continuous, as it was supposed to be, and could not have been relied on to serve as a cut-off, as for levee construction or flood protection. A head of water against such a discontinuous non-impervious cut-off wall would lead to piping and failure of the levee.

The author has made a comprehensive and valuable review of many other developments in the field of grouting. Especially interesting are his disclosures on the results of fluidity increasing with vigorous and prolonged mixing and mixing with a colloid mill. The writer would like to ask Kennedy to describe the "Charlotte" colloid mill.

The author is correct in emphasizing the difficulty of grouting medium and fine sands with cement slurry. In the writer's experience, this is attempted much too frequently and almost invariably with no successful result.

SYMPOSIUM ON GROUTING: CONSTRUCTION OF ROCKY REACH GROUTED CUTOFF[a]

Discussion by Judson P. Elston; E. S. Bell; and John E. Wagner

JUDSON P. ELSTON,[3] F. ASCE.—Under the heading "Design of Grouted Cutoff" it was interesting, and rewarding to read the following; "Grout consisting of finite particles such as cement, or clay and cement, can be injected only into material having more spaces several times the size of the coarser materials in the grout. Such grouts would not penetrate sands containing more than about 10% of material finer than 1 mm. Accordingly, it was to be expected that sand members in the alluvials, in general, would not be grouted if cement or cement-clay grouts were used alone."

The authors' description of the exploration and preliminary test work conducted and deductions arrived at during the purely design (or pre-construction) phase indicate the excellent and thorough manner by which the problem at Rocky Reach was finally resolved.

A slight difference of opinion does exist concerning the statement, "Neat cement is not satisfactory for grouting alluvials because of sedimentation or settling within the soil mass, especially within the coarser materials. Mixing of the cement with other materials is necessary to form a stable grout to fill the voids completely after it has set."

The primary purpose of clays, bentonites and fly ashes in conjunction with cements or sands, would seem to be to retain the cement and sand grains in suspension for a longer period of time before actual sedimentation develops. This occurs because of the weight of the suspending material or lower specific gravity. These suspension characteristics of certain materials also give rise to the beliefs held by some grout people that such materials through these characteristics help "lubricate" the grout. Many retarders and so-called aids to injection of grout serve much the same purpose although often their costs are higher than the more commonly used fly ashes, bentonites and diatomaceous earth materials.

Neat cement grouts, or with certain combinations of other materials, are satisfactory for grouting alluviums until the openings and voids, or pore spaces, to be filled are within several sieve sizes of the grouting material being used. The writer does not think the problem is as much settlement and sedimentation as it is a filtering action in which under pressure the fluids are actually filtered or forced out of the base mix resulting in a cement or solids "mat" up against a sand mass that has passed the water but retained the solids.

Swigers' description of difficulties experienced with the silicate grouts matches the writer's experiences with these same materials. There is, how-

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[a] April 1961, by W. F. Swiger (Proc. Paper 2796).

[3] Civ. Engr., Corps of Engrs., Rock Island Dist., Rock Isalnd, Ill.

ever, disagreement with his opinion of the chrome-lignin chemical grouts. They are considered to have excellent possibilities. This chemical sets up to approximately the hardness of a rubber truck tire, is easily controlled and handled (with care), has excellent bonding and elastic properties with rock and concrete, and from both laboratory testing and long time operation is considered to be relatively non-leaching and stable.

The author's statements concerning the undesirability of heave are sound. There has been a curious school of thought that soils can be displaced in a horizontal direction through the use of sufficient pressure and the ruptured or dislocated materials then grouted. Swiger's remark can not be repeated too often.

"It is desirable to operate at a pressure as high as can be reasonably tolerated without heaving, yet to avoid the slight over pressure that starts the movement."

The writer can not agree wholeheartedly that the width of a grouted cutoff is any criteria of its effectiveness as an impervious barrier, but this could easily become a longwinded academic discussion. Also, when considering that most of the cements manufactured today (1961) using air separation equipment are such that practically 100% pass the 100 sieve and 94-96% pass the 200 sieve, the writer is not entirely convinced that grain size is the governing criteria or problem so much as the decision as to whether to use flyash or chemical grouts in lieu thereof from an economic point of view.

In closing the writer would like to refer all engineers with an interest in foundations to Swiger's statement, "Most important of all grouting is an art." It is realized by all of us who have been close to the work over the past years, that grouting is progressing toward the day when it can honestly be called more of a science and papers such as Swiger's are bringing that day much closer to realization.

E. S. BELL,[4] A. M. ASCE.—Grouting of alluvium is usually thought of as the filling of the pores of the soil with grout. The high pressures used on many jobs with limitations imposed on the "take" of grout and on ground heave indicate that the grouting may in fact be achieved on these jobs by a process of hydraulic fracturing of the soil, followed by a filling of the cracks created by this fracturing. The experience during the grouting at Mattmark,[5] where the fourth pass during the grouting increased the foundation permeability rather than decreasing it, is an example of this effect.

May some details be given of the studies leading to the drastic reduction of grouting pressure from 200 psi to 30 psi? Is the statement that the latter grouting pressure gave an overpressure in the formation of about 75 psi intended to mean that the grout pressure at the point of injection in the soil formation was 75 psi or that this pressure was 75 psi more than the existing effective or total stresses?

JOHN E. WAGNER,[5] F. ASCE.—This comprehensive paper describing construction of the cutoff at Rocky Reach Hydroelectric Power Project and the

4 Field Soils Engr., Ontario Hydro., Toronto, Ont.

5 "Alluvium and Bedrock Grouting for Foundation of the Mattmark Dam, 7th Congress of Large Dams, Paper No. R58.

5 Capt. Corps of Engrs., Dept. of the Army, Mil. Asst. to the Director, U. S. Army Engr. Waterways Experiment Sta., Vicksburg, Miss.

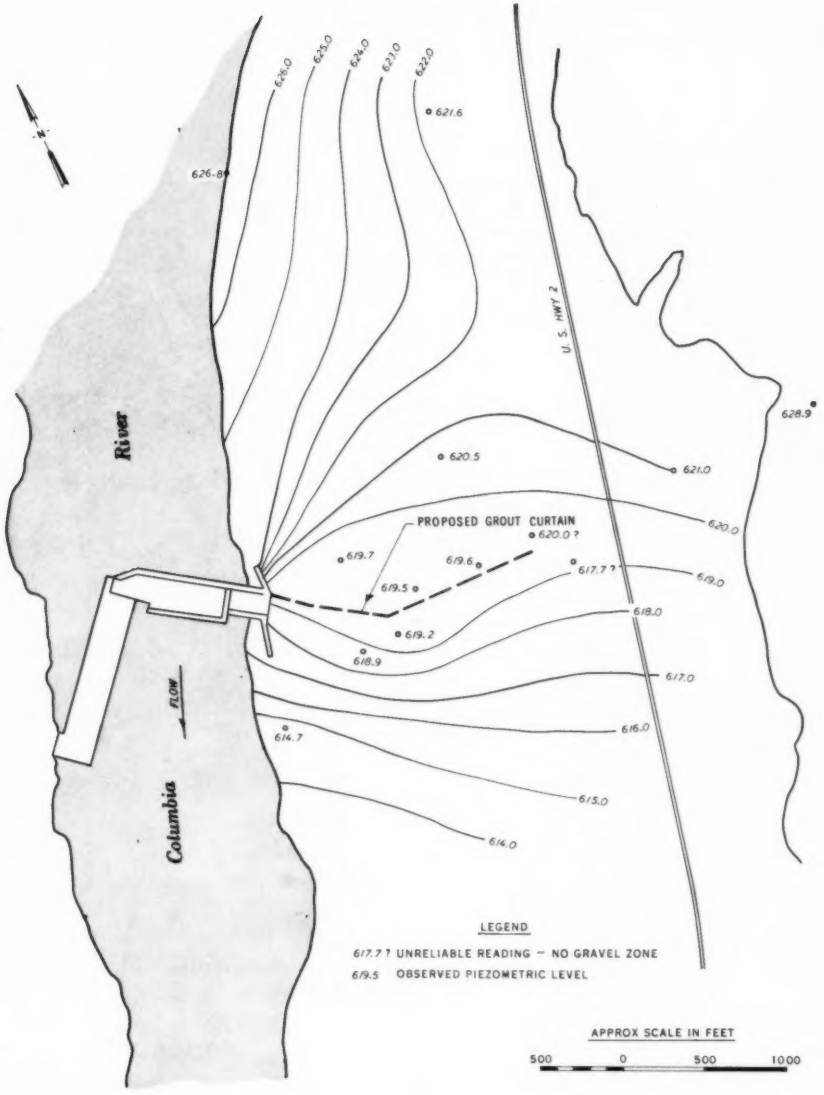

FIG. 15.—CONTOURS OF PIEZOMETRIC HEAD BEFORE CONSTRUCTION OF GROUT CURTAIN

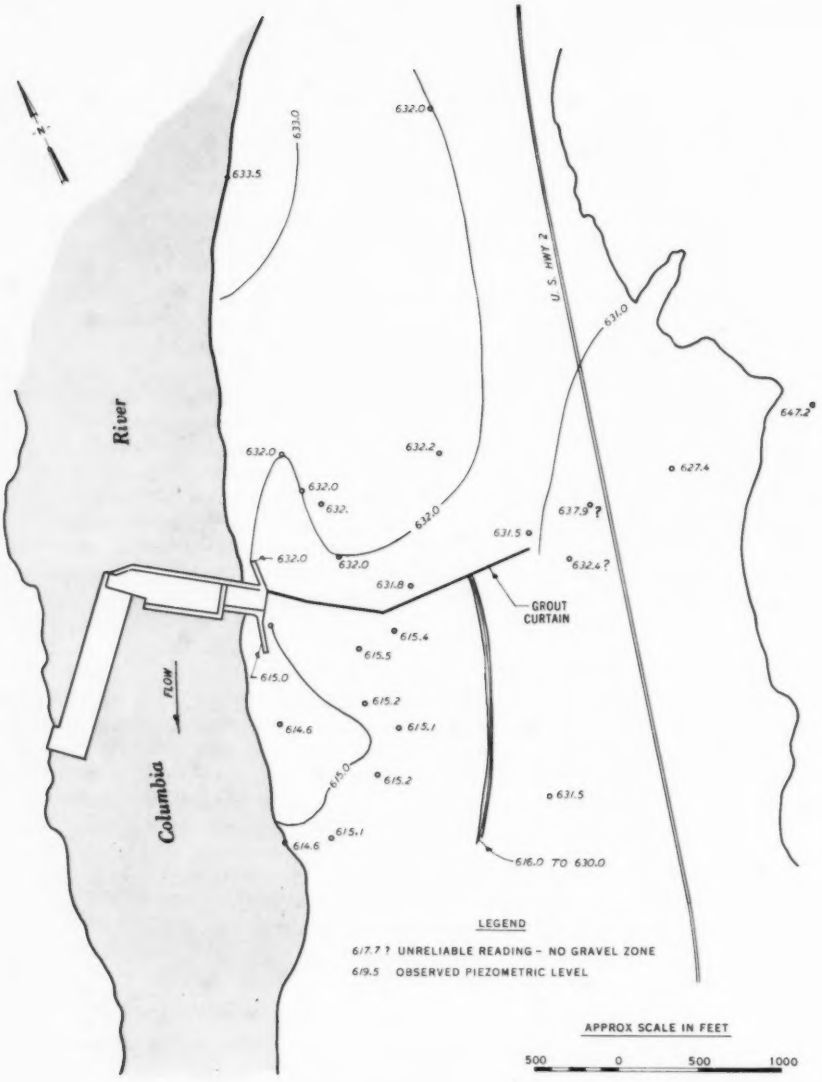

FIG. 16.—CONTOURS OF PIEZOMETRIC HEAD AFTER CONSTRUCTION OF GROUT CURTAIN

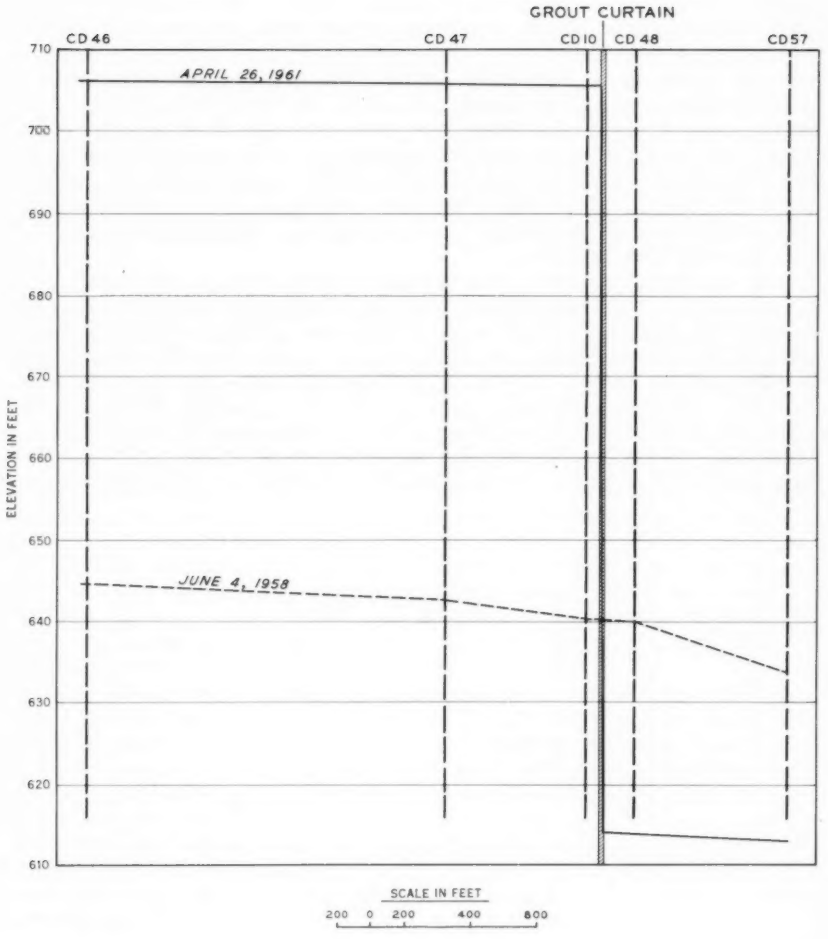

FIG. 17.—PROFILES OF PIEZOMETRIC LEVELS THROUGH THE LOWER GRAVELS

grouting and piezometer records from the project were used extensively by the writer in the preparation of a dissertation.[6]

The writer attempted to analyze the performance of the cutoff to date (1961) and to predict the future performance on the basis of construction records and piezometer observations. In order to put the piezometer records into a usuable form and eliminate erroneous readings, all water level observations were first plotted versus a calendar scale. Piezometric contours and profiles were then plotted to obtain an indication of water flow conditions through the east abutment at various stages of the construction and river conditions. From these and other studies, the performance of the cutoff was described and future performance forecasted. Fig. 15 shows contours of piezometric head in the east abutment prior to construction of the grout curtain. From this figure may be seen the gradual drop in piezometric head through the terrace; location of the cutoff is indicated by a dashed line. Fig. 16 shows piezometric contours existing in the same area after completion of the cutoff. The abrupt drop in piezometer levels across the cutoff clearly indicates the effectiveness of the grouting.

One of the objectives of the cutoff was to limit seepage sufficiently so that pressures under the downstream bank would not cause slides. The effectiveness of the cutoff in controlling seepage pressures is illustrated by the profiles of piezometric levels shown in Fig. 17. In this figure are shown piezometric profiles through the lower gravel deposit for the maximum river stage in 1958, prior to completion of the grout curtain, and for late April, 1961, after the first filling of the power pool. A comparison of these two profiles indicates a flatter gradient in the gravels downstream of the cutoff after filling of the reservoir than existed there prior to construction of the grouted cutoff.

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6 "Construction and Performance of the Grouted Cutoff, Rocky Reach Hydroelectric Power Project," by J. E. Wagner, thesis presented to the Univ. of Illinois at Urbana, in 1961, in partial fulfillment of the requirements for the degree of Doctor of Philosophy.

SYMPOSIUM ON GROUTING: GROUTING IN FLOWING WATER AND STRATIFIED DEPOSITS[a]

Discussion by Judson P. Elston

JUDSON P. ELSTON,[9] F. ASCE.—Chemicals for grouting of rock and soil foundations have been available for a number of years. Their chemical, collodial characteristics or viscosities or both, and of course, costs, have been such that their usefulness was limited to a narrow range of foundation conditions.

The writer has used the silicates and chrome lignins both in the laboratory and in the field. The chrome lignin is a relatively inexpensive material that handles easily, is susceptible to close control, and has every appearance of resisting leaching and remaining stable under observed in-service conditions. There have been objections to the use of chrome lignin, not the least important of which is the low compressive strength of the hardened material.

From a study of the paper it would appear that AM-9 has characteristics and qualities that in one respect or another have been missing from other known commercial chemical materials. The grouting pattern shown on Fig. 14 and description of the experiments using this pattern are illuminating. The laboratory experiment was tremendously scaled down from what would be needed in an actual field operation. It would be of great value to those interested in the potentialities of AM-9 for critical foundation problems, such as existed at Rocky Reach, to see the results of the laboratory experiments, as shown in Fig. 14, projected into a full-scale field operation under, of course, known foundation conditions of materials in their natural state.

So little information has been made available to the engineering profession as to the details of chemical grouting done in the past, that the paper is indeed refreshing and of great interest and value to all geologists, soils, foundation, and design engineers concerned with foundations. Unfortunately, the literature on the subject of chemical grouting, in the writer's opinion, has usually been of little value to the engineering profession.

Chemical AM-9 should have great possibilities for the future, and engineers will watch with interest the cost potential, especially as to reductions in the present price that would enable AM-9 to compete more favorably, in large quantities, with cement, fly ash, and other chemicals now available.

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[a] April 1961, by R. H. Karol and A. M. Swift (Proc. Paper 2797).

[9] Civ. Engr., Corps of Engrs., Rocky Island Distr., Rocky Island, Ill.

IMPACT WAVES IN SAND: THEORY COMPARED WITH EXPERIMENTS ON SAND COLUMNS[a]

Discussion by H. G. Hopkins; Robert E. Fulton and A. J. Hendron, Jr.; and E. T. Selig and E. Vey

H. G. HOPKINS.[19]—There was a considerable need for increased basic knowledge of the response of natural soils to transient loads. In particular, the elucidation of the effect of rate-of-strain on the mechanical behaviour of soils was a matter of fundamental importance with which the present paper was concerned. It seemed inevitable that preliminary theoretical approaches made towards the mechanical behaviour of solids must often be essentially phenomenological in nature, but nevertheless the ultimate justification of an approach had to be found in physical terms.[20] However, such justification could seldom be expected to be easily achieved, certainly in the case of natural soils whose physical characteristics were both complex and diverse. Malvern's rate-of-strain theory had originally been applied to metals, and now Blaine R. Parkin proposed its application to sands. The results presented in the paper seemed to indicate such an application could be realistic, and the writer thinks that the present approach merited further study. He wishes particularly to support Parkin's expressed views on the limitations of phenomenological approaches.

ROBERT E. FULTON,[21] A. M. ASCE and A. J. HENDRON, JR.,[22] A. M. ASCE.—As the author correctly states, his analysis is certainly not the last theoretical investigation that will use the M.I.T.[23] data as a basis for comparison when studying wave propagation in a granular medium. As such, the author's analytical interpretation of this data is another significant step toward an understanding of the fundamentals of soil dynamics.

Although others have presented dynamic studies on the behavior of sands, the M.I.T. data are certainly among the most reliable data available. Nevertheless, caution must be exercised when using static triaxial stress-strain curves with a one dimensional equation of motion in an attempt to describe the propagation of a wave down a column of sand under constant lateral stress.

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[a] June 1961, by Blaine R. Parkin (Proc. Paper 2828).

19 Armament Research and Development Establishment, Fort Halstead, Sevenoaks, Kent, England.

20 "Dynamic Anelastic Deformations of Metals," by H. G. Hopkins, Applied Mechanics Reviews, June, 1961.

21 Asst. Prof., Dept. of Civ. Engrg., Univ. of Illinois, Urbana, Ill.

22 Research Assoc., Dept. of Civ. Engrg., Univ. of Illinois, Urbana, Ill.

23 "The Behavior of Soils under Dynamic Loadings: 3. Final Report of Laboratory Studies," by R. V. Whitman et al, M. I. T. Report, Dept. of Civ. and San. Engrg., August, 1954.

The main inconsistency is that there are lateral deformations and, thus, lateral inertia effects in a sample under constant lateral stress that are ignored by the one-dimensional equation of motion.

It is significant that in static tests presented in the 1954 M.I.T. Report on the same Ottawa Sand, the lateral strains were of the same order of magnitude as the axial strains. Hence, there is little doubt that lateral inertia effects or, more specifically, the boundary conditions of the two dimensional problem had a major bearing on the behavior of the sand. It does not seem logical to discard the lateral inertia phenomena that are known to exist in reality for a strain rate mechanism whose significance is questionable. The 1954 M.I.T. Report in which the wave studies were presented also contained a study that was made specifically to investigate the strain rate effect in sands. The results were inconclusive; however, it is quite likely that some small effects exist. If it is established that strain rate effects are significant in sands, no doubt they should be included in such an analysis, but at the present it does not seem reasonable to ignore lateral inertia effects in favor of an arbitrary strain rate effect or some other conjectural mechanism.

This discussion is not meant to discredit the author's study because it is recognized that the theoretical solution to the two dimensional problem that includes lateral inertia is in itself a formidable task. Hence, the solution to the one-dimensional problem should be the first step. It is, however, suggested that closer attention should be given to a correlation of the boundary conditions of the experiment with the equations of motion and static stress strain curve used for theoretical comparison. The significance of the boundary effects on the behavior of the soil is emphasized in such an analysis as presented by the author. If the behavior is in fact one-dimensional, as the theory presumes, the radial displacement should be zero, and the stress strain curve for sand would be concave upward rather than concave downward. Such an effect would have a major bearing on analytical studies similar to the author's.

The recent investigations in soil dynamics point out the need for a great deal of effort to be expended toward defining the interrelation between the true stress-strain properties of a soil and the boundary conditions. At present, there is not a true stress-strain relation defined for any soil, not even a dry sand. This fact results in part from the omission of a thorough study of a particular type of test before moving on to the extraction of data describing the behavior of soil under a different set of boundary conditions. There is known only the behavior under certain boundary conditions—some of which are actually difficult to define—and it is not possible to predict from this knowledge what phenomenon would be manifested under a different set of boundary conditions. In fact, until the static stress-strain relations for soils are defined, it is quite unlikely that any fundamental advances will be made in defining the dynamic behavior. To date (1961), the differential stress-strain relations derived by Mindlin and his co-workers[24] to describe a granular medium seems to be the best approach. It is, unfortunately, limited to a dry material composed of equi-radii spherical particles, but it is at least consistent with the fundamentals of mechanics and is a good starting point for the development of a more general theory.

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24 "Mechanics of Granular Matter," by H. Deresiewicz, Advances in Applied Mechanics, Vol. 5, Academic Press, Inc., New York, N. Y., 1958, p. 233.

In the 1954 M.I.T. Report, R. V. Whitman discussed the aims toward which fundamental stress-strain studies in soils should be directed and suggested the need for more test data, under the following three condtions:

1. Compression under a hydrostatic stress state;
2. axial compression with constant lateral stress, such as the triaxial test; and
3. one dimensional compression (i.e., strains prevented in two perpendicular directions).

Whereas all of these stress states are important, it would appear that the first of these that should be investigated would be the one-dimensional state,

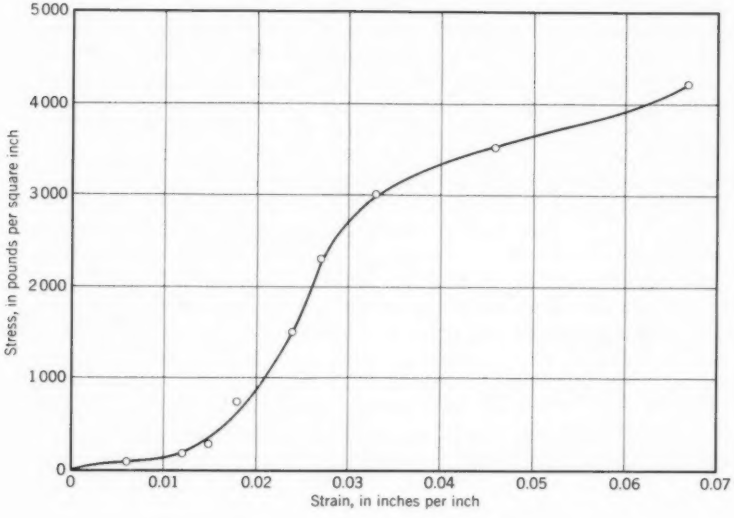

FIG. 13.—CONFINED STRESS STRAIN CURVE FOR UNIFORM DRY SAND (AFTER ROBERTS AND DE SOUZA)

particularly because interest in underground nuclear effects during recent years has resulted in the availability of a fairly large amount of literature on the subject. However, even here there are many problems involved in obtaining a true one-dimensional static stress-stain curve for a material such as sand. Among the many requirements necessary are those of applying vertical and lateral loads in such a fashion as to allow no lateral strains. A simple calculation will show that if the applied stress on sand reaches into the hundreds of psi, even an infinitely large steel container is not sufficiently rigid to ensure a minimum amount of lateral strain.

Hence, the loading sequence must be that of a horizontal combined with a vertical stress. Coupled with this is also the requirement that no restraint be provided to the soil by the loading mechanism that will interfere with the

movement along the edges of the sample. To satisfy all these requirements is no small task.

One of the best attempts to the static problem was carried out by W. Kjellman,[25] who tested sand under numerous states of stress, including one dimensional, triaxial, and hydrostatic. Unfortunately, the loading apparatus was quite intricate, and only a few experimental results were reported.

In some more recent results from M.I.T., J. E. Roberts and J. M. de Souza[26] give some data for the stress strain relation for static tests on sand up into the high pressure regions. These results have been replotted by the writers (Fig 13) in the standard stress-strain curve format, in order to observe the shape of the curve. This data was determined from sand confined in a consolidometer ring that was used to restrict lateral displacement.

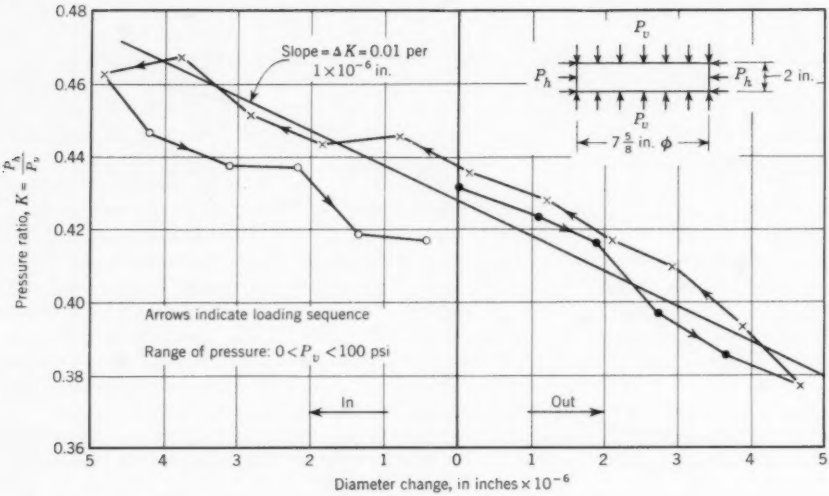

FIG. 14.—EFFECT OF LATERAL MOTION ON PRESSURE RATIO K FOR DRY SAND, VOID RATION = 0.77 (AFTER SPEER)

Such results might naturally be considered fairly reliable as one dimensional results. This is not the case, however, except in the low pressure regions. Whereas the shape of the curve may be correct in these regions, the breaking point is questionable in the high pressure regions, due to strains in the container.

Some idea of the major effect that lateral restraint has on the behavior of the sand is shown by some previously unpublished results of T. L. Speer,[27]

25 "Report on an Apparatus for Consummate Investigation of the Mechanical Properties of Soils," by W. Kjellman, First Internatl. Conf. on Soil Mechanics and Foundation Engrg., Cambridge, 1936, Vol. 2, p. 16.

26 "The Compressibility of Sands," by J. E. Roberts and J. M. de Souza, Proceedings, ASTM, Vol. 58, 1958, p. 1269.

27 T. L. Speer, Personal files of R. B. Beck, Prof. of Foundation Engrg., Univ. of Illinois, 1944.

given in Fig. 14. He utilized a flat cylindrical specimen (7 5/8 in. in diameter by 2 in. high) and a container with which he could increase axial and radial stresses in such a manner that the radial strain would be zero. The apparatus was also used to study the variation in lateral restraining pressure with measured lateral strains. It was shown that with only a small amount of lateral movement the lateral resistance required decreased considerably. Hence, only a small amount of lateral strain is necessary for the soil to exhibit phenomena that is not within the one dimensional class of problems. Thus, the rigidity of the container is quite significant.

The writers currently are conducting a testing program to obtain a one dimensional static stress-strain curve for sand in the high pressure regions.

The purpose of this discussion is not to detract from the excellent work of the author but to call attention to the need for more fundamental studies in the behavior of soil. To the writers' knowledge, there is not reliable data available that describe a truly one-dimensional stress-strain curve for standard Ottawa sand. Studies continue, however, with the ambiguous but, unfortunately, necessary practice of attempting to correlate one-dimensional dynamic analyses with the available static results made under totally different boundary conditions. That there is difficulty in making correlations should not be surprising. In fact, when correlations do exist one should be wary of the results. As one must walk before he runs, so must the static problem be solved before a true understanding of the dynamic problem may be resolved.

E. T. SELIG,[28] A. M. ASCE, and E. VEY,[29] F. ASCE.—The purpose of the study reported in the paper was to establish whether excluding effects of lateral inertia, a theory could be derived to account for observed wave propagation phenomena in a column of sand. The proposed theory is based on a specific constitutive relationship, essentially a plastic strain-rate effect, and suitable representation of the actual experimental details of some MIT research. The author should be commended for his thorough analysis, made especially difficult due to the necessity of considering experimental details of work with which he was not associated.

The purpose herein is to comment on several of the assumptions used by Parkin to represent the experimental data. The M.I.T. investigators are, of course, in the best position to judge the appropriateness of these assumptions describing their work. However, it is felt that recent experimental studies of a similar nature conducted at Illinois Institute of Technology under a National Science Foundation (NSF) grant may provide a basis for a reconsideration of some of the assumptions.

The IIT studies included tests in which 3-in. diameter, 4-in. and 8-in.-long, specimens of dry Ottawa sand were impacted with a solid 3-in-diameter steel pendulum, the sample being mounted on a second similar pendulum (Fig. 15). The pendulums were constructed in order that accurate pressure-time records for the two ends of the specimen could be obtained from the pendulum acceleration records during the impact. Thus, the pendulums acted as completely rigid pressure gages providing records analogous to those shown in Figs. 5, 6, and 9.

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28 Assoc. Research Engr., Soil Mechanics, Armour Research Foundation of Illinois Inst. of Tech., Chicago, Ill.

29 Prof. of Civ. Engrg., Illinois Inst. of Tech., Chicago, Ill.

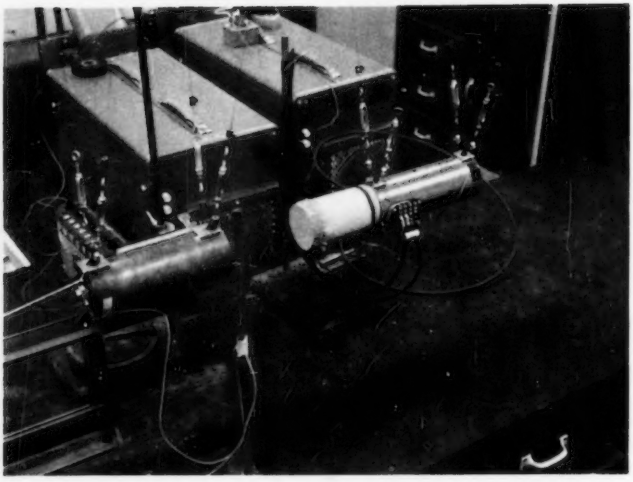
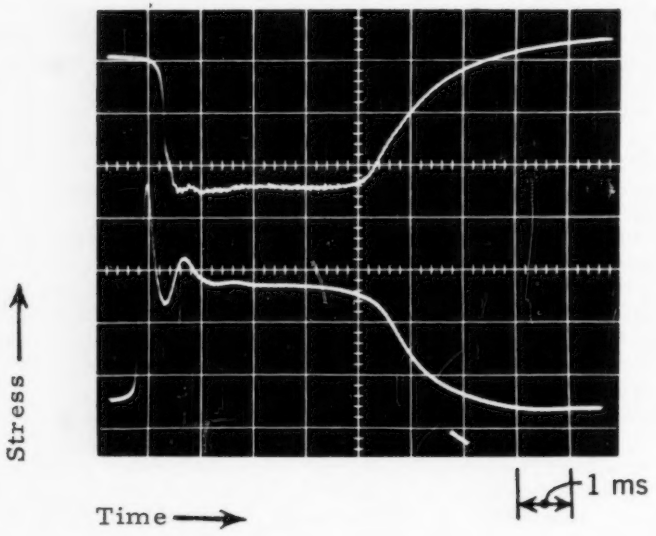


FIG. 15.—PENDULUM ARRANGEMENT FOR IMPACT TESTS



Top: Reaction end maximum stress= 32 psi
Bottom: Impact end peak stress = 62 psi,
ultimate stress= 32 psi

FIG. 16.—IMPACT TESTS ON 20-40 OTTAWA SAND SPECIMEN

A representative record is shown in Fig. 16, in which the void ratio was 0.50, the confining pressure 12.2 psi, and the specimen length 8 in. At the impact end of the specimen, the initial velocity was 37 ips and the displacement, stress, and acceleration were zero. At the reaction end, the initial velocity, displacement, acceleration, and stress were also zero. The general shape of these two stress-time records in similar to those obtained in the MIT experiments. In particular, there is an initial spike on the impact end followed by two smaller oscillations of stress. The magnitude of this spike varied considerably with void ratio, confining stress, and impact velocity.

In matching the theory to the MIT data, the author assumed that the gages were flexible, thus creating the condition that the initial velocity at the impact end was zero. Parameters s and k were then chosen so that the theoretical and experimental stress magnitudes were identical for the initial part of the pulse. Also, it was stated that the ram and impact-end gage separated immediately after impact, and, hence, the author concluded that the gage was pushed into the sand with an initial velocity V_0 imparted to it by the ram. Under these conditions the impulse associated with the initial portion of stress-time record was found to be considerably greater than the change in momentum of the gage. This discrepancy was thought to be due to an error in measurement of the V_0 so that in the analysis a corrected velocity V_0' was used, that made the impulse and momentum change equal.

There are valid arguments in favor of the preceding conclusions. However, a consideration of the IIT results suggests that the conditions may have been otherwise. It is felt that in the MIT tests the initial velocity at the end of the specimen was not actually zero, but was closer to that of the gage (V_0). Because the sand column is compressible, this condition does not require that the impact-end stress start discontinuously with a non-zero value, as may be seen from Fig. 16. Although the ram and gage undoubtedly separated soon after impact, it could not have done so without transmitting part of its momentum to the specimen, because the gage was initially in contact with the specimen. As a result, the change in momentum of the gage would have to be less than the impulse given by the stress-time curve. This would explain the observed discrepancy. Of course, there is still the possibility of an error in the measurement of V_0.

One of the purposes of the IIT experiments was to provide data under more precisely known boundary conditions. A considerable body of data has been obtained from these experiments. The results show that characteristics similar to those observed in the MIT tests may be obtained with an initial velocity at the impact end, whereas the stress, displacement, and acceleration are zero. If the Parkin phenomenological model may be made to fit this data, then its validity will be strengthened.

There still remains to be determined a physical explanation for the observed strain-rate effect implied in the theory, in order to understand the behavior of the sand at the impact end. An explanation that seems to fit the experimental observations is that the magnitude of stress produces failure in the soil. This failure is confined to the soil near the end of the specimen and involves some movement of particles and lateral yielding. The stress that is propagated through the specimen is the lower value reached after the sudden drop in stress level. In some very short specimens, a peak was also obtained on the reaction end of the specimen. Because of the short travel in this case, there was little reduction in stress as the wave propagated through the specimen,

and the magnitude that occurred when the reaction pendulum was encountered was sufficient to again create local failure. In certain tests in which a high vacuum was used with low impacting velocity, there was no peak in the stress curve, indicating that in this case the stress level did not reach the failure value even at the impact end.

On the basis of the preceding explanation, it is obvious that as far as stress propagation is concerned, the peak stress value is of little consequence, and only the stress attained after the sudden drop occurs is actually propagated through the soil. Nevertheless, it is interesting in understanding the dynamic soil behavior to be able to explain this phenomenon.

UNCERTAINTIES IN EARTH DAM DESIGN[a]

Discussion by Harry R. Cedergren; R. C. Harlan; and Charles S. Hedges

HARRY R. CEDERGREN,[12] M. ASCE.—John M. Bird, M. ASCE, and the United States Army Engineers should be given a hearty "thank you" for presenting to the engineering profession the lines of thought that led to the design of two important dams. Both dams have been designed with vertical "chimney" type drains, which are a comparatively recent innovation in earth dam design. There is a distinction between a chimney drain and a protective filter between zones of highly different gradation and permeability. The transition (filter) serves to protect a core against erosion into the large voids of a shell. The chimney drain may also serve as a transition, but its principal purpose is to remove water and stabilize the downstream part of a dam. In the 1930's and early 1940's designs often provided horizontal drainage blankets under the downstream portion of dams of otherwise "homogeneous" cross section. In 1942 a chimney type drain was developed for a homogeneous earth dam that was to be constructed of a silty loam soil, moderately variable in the borrow area. No large deposits of granular materials were available in the vicinity of the dam site (the Neal Creek Dam site, in Oregon); therefore, this dam could be designed only as a homogeneous dam. Flow-net studies by the writer demonstrated that the construction of this dam entirely out of one class of material, with a conventional horizontal drainage blanket would not eliminate the possibility of seepage outcropping on the downstream slope. This uncertainty of construction conceivably could have led to an important loss in stability under long periods of full reservoir. To overcome this uncertainty, the dam was designed as a homogenous section with the exception that a more-or-less vertical screen or chimney of clean sand was stipulated slightly downstream from centerline. The chimney drain guaranteed a low saturation line, regardless of the quantity of stratification that might be built into the dam.

The chimney principle also provides protection against the uncertainty of adequate permeability of granular shell material that may, as in the examples cited by the author, be little or no more pervious than the "impervious" core section. The inclusion of gravel or boulder sizes in a soil-gravel mixture may not result in an increase in permeability over that of the matrix soil. A large retaining wall that collapsed was found to have been constructed with a "pervious" backfill that graded from 3-in. rock down to approximately 12% of minus 200 material. The mixture looked definitely granular, but it was nearly as dense as concrete and approximately as impervious. Naturally occurring river gravels in many parts of the country contain substantial clay or silt and are not

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a June 1961, by John M. Bird (Proc. Paper 2832).

12 Senior Materials and Research Engineer, California Division of Highways, Sacramento, California.

very pervious. When these materials are used in the outer sections of dams, the chimney principle removes the risks due to the uncertain permeability of the shell material.

By means of flow-nets the advantages of the chimney type of drain for earth dams is demonstrable. For example, Fig. 14 shows the theoretical position of the saturation line in Terminus Dam for several conditions. It compares the favorable position it would have when a chimney drain is provided, with positions it could have without the drain. The effectiveness of the drain in combatting embankment stratification is evident.

A factor that can nullify all the good intentions of expert designers is a break-down in the conveyance of vital design criteria from the design engineers to the construction engineers. Well prepared plans and specifications normally will insure that construction is handled in a manner that will assure a dam with properties approximately equivalent to those contemplated by the designer, but

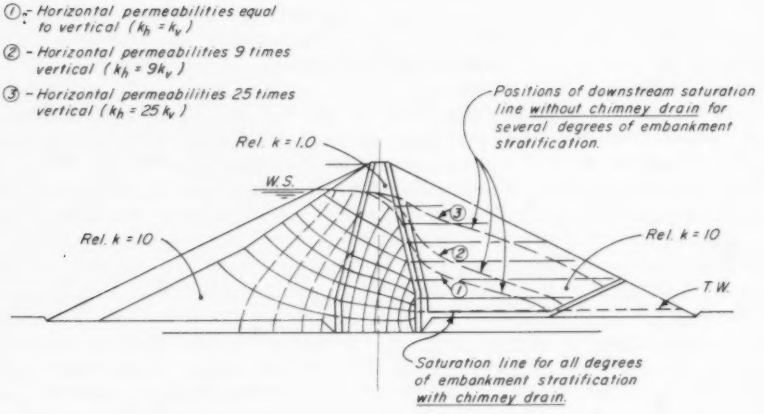
① - Horizontal permeabilities equal to vertical ($k_h = k_v$)
② - Horizontal permeabilities 9 times vertical ($k_h = 9k_v$)
③ - Horizontal permeabilities 25 times vertical ($k_h = 25k_v$)
Positions of downstream saturation line without chimney drain for several degrees of embankment stratification.
Rel. k = 1.0
W.S.
Rel. k = 10
Rel. k = 10
T.W.
Saturation line for all degrees of embankment stratification with chimney drain.

FIG. 14.—BENEFITS OF CHIMNEY DRAIN FOR CONTROLLING SATURATION IN TERMINUS DAM

exceptions do occur. In organizations in which entirely separate staffs design dams, and other staffs supervise their construction, there is a chance that an important assumption of the designer may be overlooked in the construction. To cite an example, the writer once visited an earth dam being constructed in a manner that virtually guaranteed its failure after it was put in service. The dam was being constructed with fine wind-blown silt on a foundation of the same kind of soil. An internationally known earth dam consultant who knew the risks of building a dam under these conditions had designed a large cut-off trench with a graded pervious drainage layer at the downstream side. The design was basically adequate; however, the drainage blanket was being constructed in such a manner that the coarse sizes rolled to the outside of the blanket and the fines were all on the inside. The silt fill was being placed directly against a sloping zone of boulders. The design engineer never would have condoned this construction, but the field forces were not aware of the critical condition that

they were building into this dam. The "piping ratio" of the boulders with respect to the compacted silt was more than 1,000, in contrast with a normal ratio of 5 or, perhaps, 10. This dam was completed in the manner described. Upon the first filling of the reservoir, severe internal erosion produced a seepage effluent containing the maximum quantity of silt that the water would carry. In a few days hundreds of cubic yards of embankment soil were washed out through the drain. Subsequently large sums were expended in grouting the boulders and in constructing a new drainage system. The case is perhaps a rare example, but it illustrates an uncertainty in design that may occur when there is a break-down in the chain of thought from design to construction. This uncertainty may be eliminated by having one or more experienced soils men stay with important projects from the explorations stage, through design, and on to the end of construction.

R. C. HARLAN,[13] M. ASCE.—The examination of some of the uncertainties that confront the designer of embankment dams as presented by the author is of great interest. Of equal value is the presentation of the evolution of the designs of Terminus and New Hogan Dams as more facts became available to the engineers. Too often in the literature of dams, a glowing description of the final structure is presented without any hint as to the laborious process required to achieve the successful design. In the case of Terminus Dam, John M. Bird, M. ASCE, has also been able to provide comment on the "proof of-the-pudding," the actual construction conditions.

In the conclusions, under the heading "Uniformity of Borrow Materials," the author has urged that "the designer should not place too much dependence on the uniformity of the borrow materials." Keeping in mind the economic factors that play a large part in the feasibility of embankment dams, it would be well to consider this point further. Failure to develop a design that provides for maximum use of material excavated from any source, but particularly from borrow area or quarry, inevitably results in higher cost of construction. Every load of potential fill material that is picked up from the borrow area or quarry should be placed somewhere in the dam.

Where wide variations in borrow materials are discovered during the design phase it is imperative to design the structure so that the full range of material properties will be efficiently accommodated. Under such circumstances it will be more economical to provide more generous quantities and utilize all the available material than to cut down on the bulk of the embankment and rely on considerable selection. This is not intended to suggest that the engineer must accept the worst material characteristic as a design value for any given analysis, but to point out that over-reliance on selection, waste, and construction control, as a method of assuring material quality, will usually result in higher actual construction cost.

In the description of the New Hogan Dam design, under the heading "Investigation of Terrace Gravels" valuable and interesting data have been presented on an unusual material. Certainly the selection of a different material for the main body of fill at New Hogan Dam was indicated. However, some engineers who have an opportunity to visit the New Hogan Project or study the plans and specifications for it, will ponder whether the designers have substituted one "uncertainty" for another.

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13 Senior Engr., Hydro Dept., Bechtel Corp., San Francisco, Calif.

The greatest uncertainty that confronts and often confounds the designers of rockfill structures is the proper evaluation of the nature of the material that will result from quarry operations at a given site. I. C. Steele has described the investigations necessary for proper quarry evaluation.[14] J. P. Galloway emphasized the importance of the quality of rockfill.[15] The Corps of Engineers Manual, Geological Investigations, recommends:

> "Where large quantities of rock for slope protection or rock-fill are required from unproven sources, experimental quarry operations to determine the breaking qualities of the rock should be conducted."[16]

However, the only evidence presented by Bird or by the plans and specifications for the project, as to the character of the proposed quarry at New Hogan, was obtained from eleven cored drill holes. The author described the rock and stated that it is fractured below the weathered zone. For evidence as to the depth of weathering, the writer studied the logs for the quarry drill holes. Appreciable weathering is noted for vertical depths of from 15 ft to 41 ft with an average of approximately 26 ft for the eleven drill holes. Over the area of the quarry the quantity of weathered material probably not useable in the dam as designed would amount, by the writer's estimate, to some 1,200,000 cu yd.

The designers of New Hogan Dam have estimated quarry stripping, defined as the quantity of excavation required to expose sound rock, to be only approximately 14% of the volume of weathered material previously estimated by the writer. It would be interesting to learn the scope of further investigations undertaken to ascertain the quality of a quarry site that appears questionable to the casual observer.

Experience in weathered igneous and metamorphic rocks in the higher elevations of the Sierra Nevada Mountains have shown that quarry sites showing appreciable weathering in cored drill holes are likely to be questionable as sources of rock-fill; indeed, the product is more likely to be "rocky dirt" than "dirty rock." The lack of gentleness on the part of modern construction equipment in handling rock is not always appreciated. Even with a minimum of handling, the action of blasting, digging, loading, dumping, spreading and rolling are often enough to reduce even strong though weathered rocks to a soil mass.

Interpretation of data and cores obtained from drill holes in soft or weathered formations and the prediction of the effects of excavating equipment on such formations requires the nicest judgment on the part of the engineering geologist. The percentage of core recovery should not be considered an adequate indication of the quality of the quarry site. The modern hydraulic feed rotary drilling rig using a well-designed core barrel and bit can achieve nearly complete recovery in formations too soft to be usable as sources of rockfill.

As an example, at the site of Camanche Dam, to be built on the Mokelumne River by the East Bay Municipal Utility District approximately 10 miles west

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14 "Rock-Fill Dams," by J. C. Steele, Handbook of Applied Hydraulics, C. V. Davis, Ed. 2nd Editon, 1952.

15 "The Design of Rock-Fill Dams," by J. D. Galloway, Transactions, ASCE, Vol. 104, 1939, p.1.

16 "Geological Investigations," U. S. Army, Corps of Engrs., Manual EM 1110-1801, November 1, 1960.

of the New Hogan Damsite, 98% core recovery was obtained in diamond drilling through material that will be excavated for the spillway and placed in the embankment. Investigations were made by test excavation to show whether the semi-indurated sedimentary formation could be reduced to a soil mass with ordinary construction procedures. The formation was ripped with a two-toothed hydraulic ripper mounted on a D-9 tractor, loaded with a tractor shovel into dump trucks, hauled to a test embankment area, spread, watered and rolled with a heavy tamping roller. The result was a nearly complete breakdown into a silty sand, forming a semi-pervious fill well adapted to embankment construction.

When the writer visited the New Hogan Damsite in July 1961, the quarry stripping was well advanced and it was already apparent that quarry stripping would far exceed the original estimate. In view of the significant quantity of quarry stripping now apparent, would it not have been desirable to provide for the use of the weathered rock in the embankment section and so have avoided one more uncertainty? The designers of New Hogan Dam might have taken a clue from their experience at Terminus Dam where the author states in discussing the rock obtained from the spillway excavation that, "The rock contained somewhat more files and was smaller than anticipated."

The slopes of the New Hogan Dam are relatively steep for a near vertical central core dam, therefore, those charged with construction supervision are well advised to insist on only high quality rock-fill material for the embankment.

The only similar dam with slopes steeper than New Hogan known to the writer is the upper 200 ft of the Goeschenen Dam in Switzerland.[17,18,19] Exhaustive tests on the rockfill material for that project included test fills and large diameter triaxial tests at relatively high lateral pressures.[20,21] The designers of Goeschenen and also those of Furnas Dam[22] investigated the strength of the proposed rock-fill materials by means of large diameter triaxial tests and so reduced the uncertainty connected with those materials. It is interesting to note that only the higher values obtained in tests for Goeschenen equal the 40° assumed for New Hogan without tests and that the design value used for the rockfill at Furnas is 38°.

From Bird's comments under the heading "Final Refinements" the writer infers that the designers of New Hogan considered the strength of the rock-fill material somewhat questionable because they reduced the design value of the angle of internal friction from 45° to 40°. It is interesting to note that at appar-

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17 "Discussion by J. B. Cooke on "Brownlee Sloping Core Dam," by Torald Mundal, Transactions, ASCE, Vol. 125, Part II, 1960, p. 575.

18 "Goeschenenalp Rockfill Dam Project, Switzerland," by W. Eggenberger, Proceedings, 3d Internatl. Conf. on Soil Mechanics and Foundations Engrg., Vol. 3, 1953.

19 "Construction Methods for Goeschenen Dam," by Harold J. McKeever, World Construction, September, 1960.

20 "Shear Strength of the Materials for the Supporting Shell of the Goeschenenalp Dam, Switzerland," by J. Zeller and R. Wulliman, Proceedings, 4th Internatl. Conf. on Soil Mechanics and Foundations Engrg., Vol. 2, 1957.

21 "Test Fill with Coarse Shell Material at Goschenenalp Dam, Switzerland," by J. Zeller and H. Zeindler, Proceedings, 4th Internatl. Conf. on Soil Mechanics and Foundations Engrg., Vol. 2, 1957.

22 "Brownlee Sloping Core Dam," by Torald Mundal, Transactions, ASCE, Vol. 125, Part II, 1960, p. 520.

ently the same time they steepened the downstream slope from 1 on 1.75 to 1 on 1.6.

A description of stability analyses performed on the final designed section and the computed factors of safety would have added to the interest of the paper, although the nature of the sliding wedge method of stability analysis is such that for an embankment section of this type it provides little more than a check on the assumed strength of the rock-fill material.

Bird has described the strength testing of the core material at some length, but is is the rock-fill shells that provide the stability for the dam. New Hogan Dam is another example of the "great hydraulic principle" first enunciated by George L. Dillman in 1912.[23] "Construct one impervious surface, and build the rest of the structure to support that surface." The rock-fill in the case of New Hogan Dam is the support for the impervious surface; therefore, the strength of the rock-fill deserves as much consideration as the strength of the core.

Another interesting aspect of the New Hogan design is that according to the specifications the upstream shell and core are to be founded on weathered rock that has been subjected to a foundation cleanup including extra excavation where directed, whereas the downstream shell, which will impose higher stresses on the foundation because of its steeper slope, is to be supported on the weathered rock without cleanup. At the site it appears that this provision has resulted in a much lower foundation level for the core and upstream shell than for the downstream shell. It would be interesting to learn what steps, if any, will be taken to ascertain whether the weathered rock foundation under the downstream shell meets the design criteria of being much stronger than the embankment placed upon it.

The reduction of uncertainties in dam design is the aim of every conscientious designer. Bird should be congratulated for having described and frankly illustrated the treatment of some of these uncertainties.

CHARLES S. HEDGES,[24] A. M. ASCE.—Until recently, 1925 to 1930, all earthfill dams were designed by empirical methods. The results of these empirical designs have a history including many failures. The author infers in his introduction that the design of high dams is a design depending on empirical methods. The writer does not agree, because the design of a high dam (400 ft and up) with experience as a design criteria will be stable only through the results of the proverbial "Act of God." Experience should play two roles in the design; that of assisting the designer in a logical but rational approach in the method of analysis, and second as a cursory check of those values determined from the rational analysis.

The competent soils engineer recognizes that there are uncertainties in the methods of soil mechanics analysis for dams. The exactness of rational soil mechanics is relative, realizing that the exactness of simple arithmetic is absolute. However, the exactness of rational soil mechanics is quite adequate for the design of high dams provided the soils engineer making the design analysis is well founded in the assumptions on which the theory of soil mechanics is based.

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23 Discussion by George L. Dillman on "Construction of the Morena Rock Fill Dam, San Diego County, California," by M. M. O'Shaughnessy, Transactions, ASCE, Vol. LXXV, 1912, p. 27.

24 Soil Mechanics and Foundation Engr., Harza Engrg. Co., Chicago, Ill.

There are a number of problems in dam design lacking absolute rational approaches, however an approximate rational approach does provide information on which a good stability criteria may be based. An example of this is the method of earthquake analysis. There may never be a true rational approach to earthquake analysis, due primarily to the fact that there is no standard pattern of earthquake action. The approximate rational approach has provided dams that may and do behave in a stable manner during the most severe earthquakes.

Terminus Dam.—In the stability of the final embankment section reference is made that the embankment section is a safe-adequate structure. Preceding this, there was a discussion of field methods to reduce non-uniformities of the embankment section. From this discussion the designer decided to reduce complex construction methods and field control. Herewith a modification was needed to satisfy the stability of the embankment section. The embankment section as modified and with the use of high values of internal friction does not reflect the prudent use of conservatism but rather the expected use of good common sense.

From the section on construction notes and the self-evident fact that three of the five safety factors are below the designer's own recommended values it is seen again that safety is adequate but not conservative.

New Hogan Dam.—The development of the typical section points out the basic fallacy of the empirical design approach. This is shown by the wide difference between the assumed values of the angle of internal friction of the earthfill and the actual values. With the actual values the designer does incorporate them into the design analysis, which is good. However, at the very next point of the design analysis he assumes that the internal friction of the rockfill is 45° and apparently no physical tests are made to prove or disprove this assumption. In view of how badly the earthfill assumptions were in error, the verification of the friction value for the rockfill would have been justified. An approach to rockfill testing would be a large box shear test made in the field.

The vertical movement of the dam has been adequately considered. The present day approach to this problem is empirical, unfortunately. This property depends on reservoir filling and it is seen that some rebound does accrue upon drawdown for the first several cycles of filling and emptying. This illustrates the fact that the vertical movement is a function of the elastic or viscoelastic property of the core, primarily, and rockfill, secondarily. To this end a rational approach to the vertical movement can be developed and should be used.

Conclusions.—The uncertainities of the material properties are handled adequately in the design analysis of the two dams. With the coming of high dams (500 ft to 900 ft) the designer is being forced to develop new techniques and types of field exploration methods and laboratory tests in order to apply the rational approach to the design analysis. Also, the designer is being forced to develop new applications and refine old applications of the theory of soil mechanics to cope with the increased problems of settlements and slope stability. The designer will also be required to use prototype field tests on items of compaction methods, shear strengths, quarry methods and other problems. This is a healthy atmosphere, bearing in mind that the rational theory of soil mechanics was developed to supplant the empirical approach and its inherent failures.

Finally, the use of the nomer, conservative, is misleading, in that it is obvious from the reported values of the strength properties used in these designs and the reported safety factors that the sections are only adequate. The inference that present day methods are adequate for higher dams is also misleading. Much work has yet to be done on the methods of applying the rational theory before safety in conjunction with economy can be absolutly determined in the design of 700 ft and 800 ft dams.

SECONDARY COMPRESSION OF CLAYS[a]

Discussion by T. K. Chaplin

T. K. CHAPLIN.[19]—K. Y. Lo's paper is an interesting contribution to the understanding of secondary compression, particularly because of his extension of the Gibson-Lo model to take account of structural breakdown. It is hoped that Lo will fully clarify the following questions.

The Gibson-Lo model for secondary compression, Fig. 1(a), suggests that at high strain rates (associated with the earlier stages of consolidation), the clay ceases to stiffen up as the strain rate increases. The writer agrees that the constants a, b, and λ may be chosen to give fair agreement with experiments over approximately a 4:1 range of time in the later stages of a test. Why should that agreement mean that the model is satisfactory for extrapolating to much higher or much lower strain rates?

As Lo does not refer to the paper by A. H. Naylor and J. G. Doran[20] describing their superficially similar procedure for determining primary consolidation, could he indicate how he chooses the value of $e(\infty)$? When a clay takes a long time to complete its primary consolidation, it seems that it would be easy to confuse the primary phase with the secondary phase.

In general, there will be two $e(\infty)$ values that can give a partly linear plot of $\log_{10}[e(\infty) - e(t)]/\Delta\sigma$ against time. The first will correspond to the point of 100% primary consolidation by the Naylor and Doran method. The linearity of the curve for a clay sample generally extends from approximately 40% to 90% primary consolidation, over approximately an 8:1 range of time. The second value of $e(\infty)$ will correspond to the secondary compression, and Figs. 11 to 16 show that linearity extends over approximately a 4:1 range of time.

In many of the figures showing the determination of soil parameters, the vertical scale is so cramped that substantial curvatures could be easily missed. Could Lo please show some curves from each of Figs. 11 through 17 re-plotted with a large enough vertical scale to give slopes of approximately 45° for the linear sections? This would show whether the apparently straight sections are really straight or curved.

When comparing different methods of obtaining the point of 100% primary consolidation, it is difficult to know whether Taylor's method, Naylor and Doran's method, or Casagrande's method is best suited to clays with relatively large quantities of secondary compression, and few published comparisons are available. Could the author please show how these three methods compare on

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a August 1961, by K. Y. Lo (Proc. Paper 2885).

19 Lecturer, Dept. of Civ. Engrg., Univ. of Birmingham, Birmingham, England.

20 "Precise Determination of Primary Consolidation," by A. H. Naylor and I. G. Doran, Proceedings, 2nd Internatl. Conf. on Soil Mechanics and Foundation Engrg., Vol. 1, 1948, p. 34.

some of his typical time-settlement curves, and give his opinion on their relative merits?

Continuing from the preceding point, could Lo please indicate on Fig. 10 where he considers primary consolidation has ended for each soil type? It would seem, on the surface, that structural breakdown might occur in type III clays before even the primary consoldiation has finished and that they possess virtually no secondary compression. But the writer is sure that they can have secondary compression of the form shown.

Has the author found that the same values of the compressibility ratio b/a and the rate factor λ/b are given by consolidation tests on both very thick and very thin samples? This would be a good way of verifying the correctness of his model and the Gibson-Lo model.

Lo contrasts his methods (1) and (2) in terms of final settlement being used or not used. When a test is only taken to little more than a week (Figs. 11 to 15), a slight alteration in the estimated value of $e(\infty)$ will make a disproportionate change in the curvature of the later part of the graph.

In method (1), then, it appears to be possible to seriously affect the result by the choice of end-point. From that it would appear that method (2) is usually preferable. Could the author please comment on this, and give some typical values of the time interval $\Delta t = t_2 - t_1$ that he used in method (2) for Figs. 12, 14, 15, and 16?

Another publication[21,22] deals with primary and secondary consolidation. As these papers were published after Lo's papers were prepared, would he comment on J. Brinch Hansen's approximate theory, and particularly on the effect of pressure increment ratio that G. A. Leonards, F. ASCE, and Pablo Girault, A. M. ASCE, show to greatly affect the form of secondary compression curve of a particular clay.

It is not difficult to represent any reasonable assumed secondary compression law by a chain of Kelvin elements. This may be studied by electrical analogue[23] or by digital computation methods. The writer hopes that Lo will use chains of such elements in his future work to avoid the limitations of the Gibson-Lo model.

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21 "A Model Law for Simultaneous Primary and Secondary Consolidation," by J. Brinch Hansen, Proceedings, 5th Internatl. Conf. on Soil Mechanics and Foundation Engrg., Vol. 1, p, 133.

22 "A Study of the One-Dimensional Consolidation Test," by G. A. Leonards and Pablo Girault, Proceedings, 5th Internatl. Conf. on Soil Mechanics and Foundation Engrg., Vol. 1, p. 213

23 Discussion By T. K. Chaplin of "Pore Pressure and Suction in Soils," Butterworths, London, 1961, p. 133.

PILE HEAVE AND REDRIVING[a]

Discussion by Stephen M. Olko

STEPHEN M. OLKO,[3] M. ASCE.—Earle J. Klohn, M. ASCE, deals with the driving of 26,000 timber piles, that resulted in considerable soil displacement and pile heaving. He is to be complimented for his clear presentation and reasonable conclusions.

Steel H-piles were used adjacent to existing buildings in order to reduce the soil displacement. This was a wise decision; however, lest the impression be gained that steel H-piles are practically "non-displacement," the following case history is cited. It will show that the same general problems arise when large numbers of closely spaced H-piles are driven.

General Description.—The Cleveland works of the Jones & Laughlin Steel Corporation is located on the west bank of the Cuyahoga River in Cleveland, Ohio. As part of an expansion program, a new blast furnace was to be constructed adjacent to an existing furnace. The plan called for driving 244 steel H-piles for the furnace foundation, 141 piles for thirty-three supplementary footings around the furnace, and 726 piles for trestle structures, or a total of 1,111 piles. The foundation plan in the blast furnace region only is shown in Fig. 15.

Subsurface Conditions.—The soil conditions of the Cuyahoga River Valley have been described elsewhere.[4] At the blast furnace site a 204-ft deep dry sample boring penetrated to hardpan, retaining split spoon type samples. This boring was made to verify the stratification, because the area had been previously investigated for other construction. The boring log is shown in Fig. 16.

Typical of the area, the boring showed a surface layer of slag and miscellaneous fill underlain by approximately 8 ft of soft sandy silt. Beneath the silt are 15 ft of loose to medium dense, medium to coarse sand with traces of gravel.

Below the sand and gravel is a 120-ft thick deposit of medium to stiff glacial clay, extending to approximately El. 431 ft. This stratum is divided from El. 471 to El. 485 by a layer of stiff clay with some sand, gravel and fragments of shale, resembling glacial till. All piles were driven to approximately 100 ft and extended into this stiff layer to a tip elevation of approximately 480. Hardpan underlies the clay soils.

The average soil properties of the clays, based on tests from other borings, are given in Table 1.

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a August 1961, by Earle J. Klohn (Proc. Paper 2905).

3 Cons. Engr., 50 E. 42nd St., New York 17, N. Y.

4 "Foundation Conditions in the Cuyahoga River Valley," by R. B. Peck, Proc.-Sep. No. 513. ASCE, Vol. 80, October, 1954.

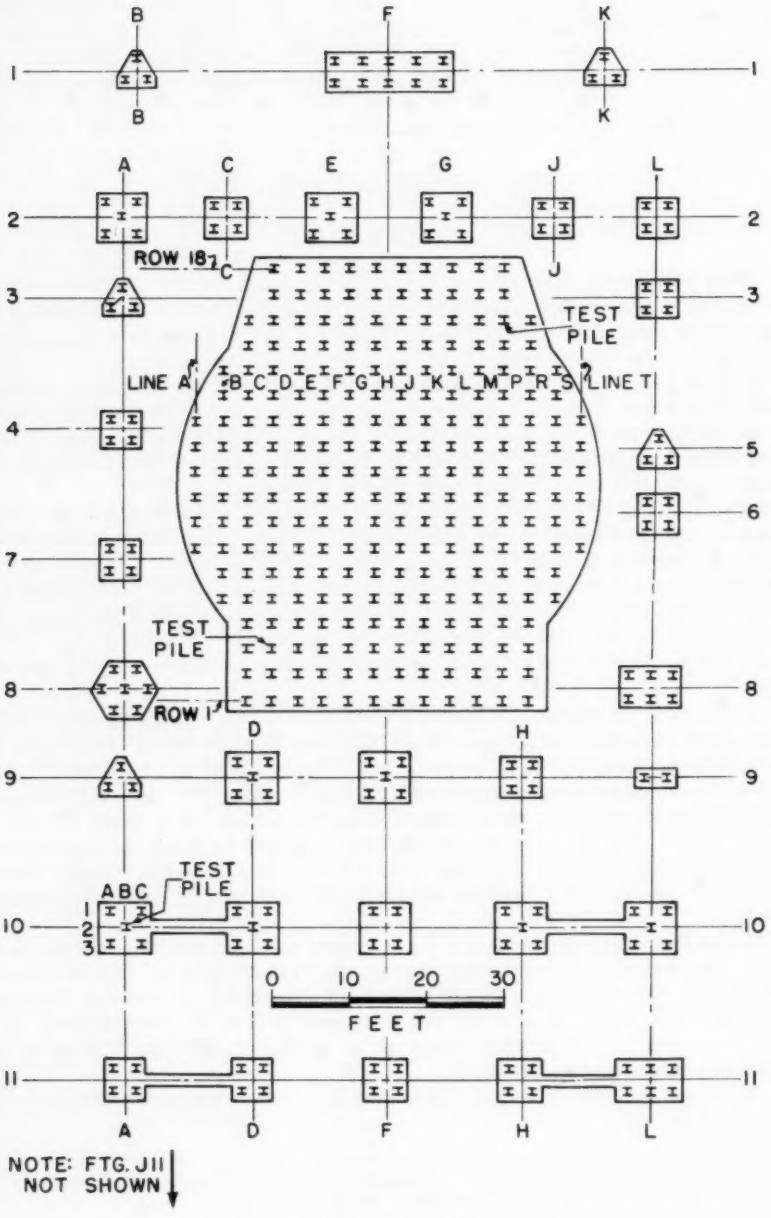

FIG. 15.—FOUNDATION PLAN BLAST FURNACE AREA

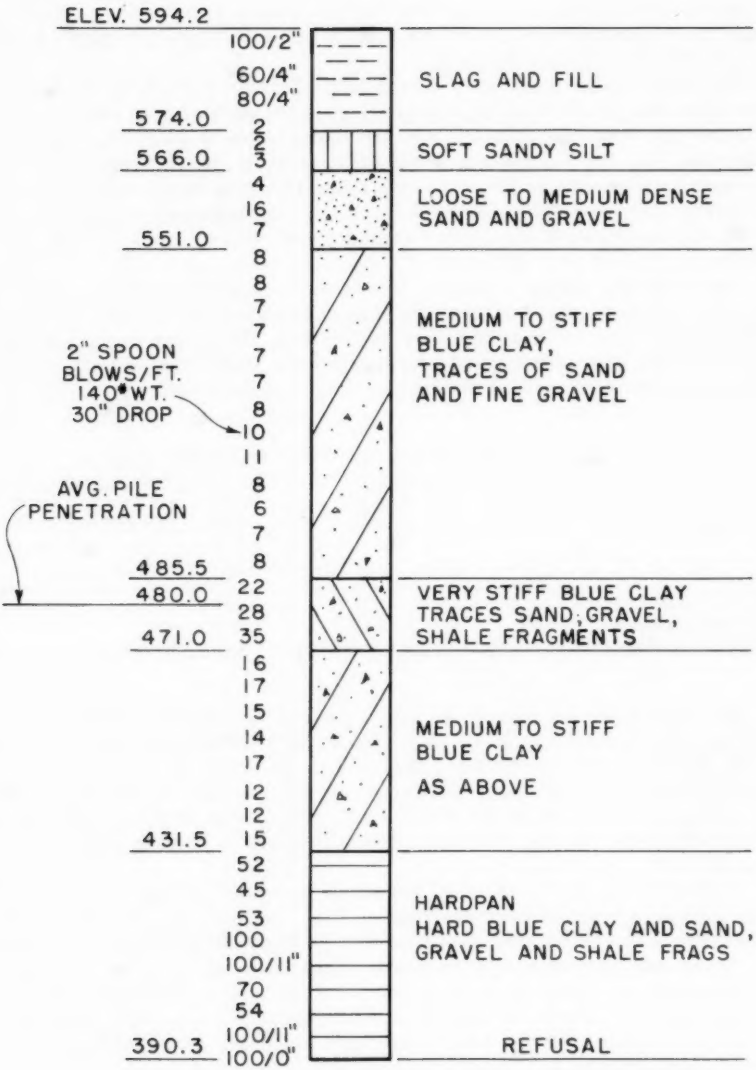

FIG. 16.—BORING LOG BLAST FURNACE AREA

Pile Load Tests.—The design of the new blast furnace foundation called for 14-in. 89 lb steel bearing piles approximately 100 ft long, loaded to 50 tons capacity. Although the shear strength of the soil was known, it was decided to verify the capacity by conducting four load tests to a maximum load of 100 tons. Two tests were made at the blast furnace foundation proper and one at each of two surrounding footings.

The piles for the tests were driven with a Vulcan 50 C. double acting hammer operating at 90 blows per minute. The same hammer was used for the entire job and therefore driving records were taken to establish a blow-count criterion. Fig. 17(a) shows the results of the driving. In all cases there was an increase in blow-counts after splicing and a return to the general pattern with redriving. An appreciable increase in blow-count was noted when the hard clay layer was encountered at a depth of approximately 100 ft. The last inch of penetration required 9 to 11 blows.

The load test set-up consisted of two reaction piles spaced 6 ft 8 in. from the test pile and joined by a cross frame. Load was applied in 10-ton increments every hour up to a total load of 100 tons. This load remained in place for 24 hr and was then reduced in decrements of 10 tons per hr. The final settlement was recorded 24 hr after all load had been removed.

TABLE 1.—AVERAGE SOIL PROPERTIES OF THE CLAYS

Clay	Natural Water Content, in %	Liquid Limit, in %	Plastic Limit, in %	Unconfined Compression Strength T per sq ft
(1)	(2)	(3)	(4)	(5)
Medium Clay	25	37	20	1.6
Very Stiff Caly	18	34	17	3.4

The results are illustrated in Fig. 17(b). The disparity between pile D-3 and the other piles is evident. This was anticipated because pile D-3 attained a resistance of only 79 blows for the last foot, whereas the other test piles required from 93 to 115 blows. Furthermore, even though the two reaction piles for test pile D-3 were driven to the same depth, they attained resistances of 95 and 101 blows for the last foot. The difference in driving resistance could be explained.

The tests showed that the piles had ample capacity to support the 50-ton design loading, based on the requirement that the maximum allowable bearing value should be one-half the test load that produces 0.005 in. of net settlement per ton of test load, deducting rebound. This would permit a settlement of 0.5 in. Actually the gross settlement, even for the worst pile, D-3, was 0.47 in. Accordingly, the design loading was considered safe for an individual pile, not taking into account the effects of long-time settlement or group action. These effects were studied separately but were not critical.

Pile Heaving.—Upon completion of the load tests, full-scale pile driving operations commenced. However, in checking test pile B-2 of footing A10 (Fig. 15) seventeen days after driving, it was noted that it had risen 1.31 in. above its original elevation. A survey of the three piles in this footing established that they were beginning to heave and that the pile closest to the driving had

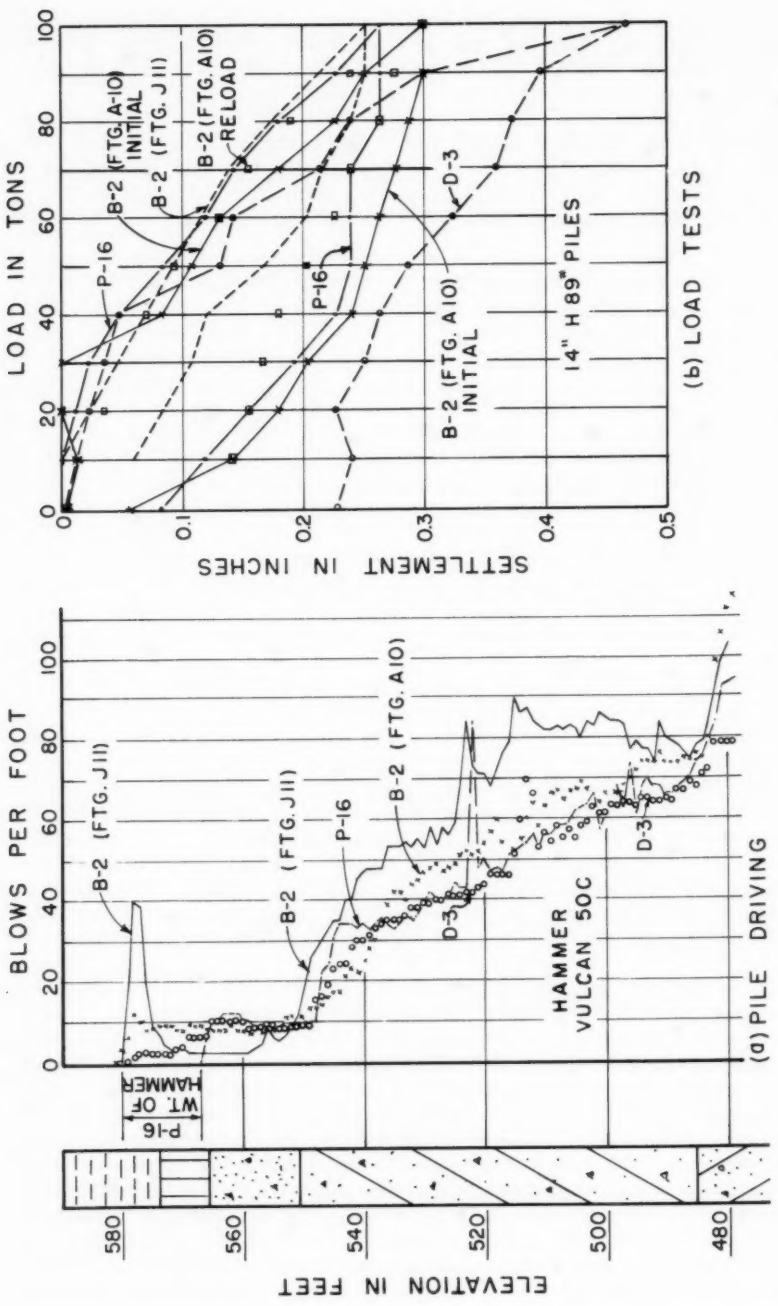

FIG. 17.—TEST PILES

come up the most. Accordingly, a reload test was performed on footing A10 to determine the loss in bearing capacity. The results of this reload test, in combination with the original load test, are shown in Fig. 17(b) and indicate the heaving had no adverse effects on the capacity of the pile.

The question arose as to whether the entire area was heaving due to either reduced overburden pressure caused by 8 ft of excavation or by volume displacement of the piles. The former seemed improbable, since the geologic history indicated that the clay soil was preconsolidated to a load of approximately 5 T/SF, which is approximately 3 T/SF above its normal load. In addi-

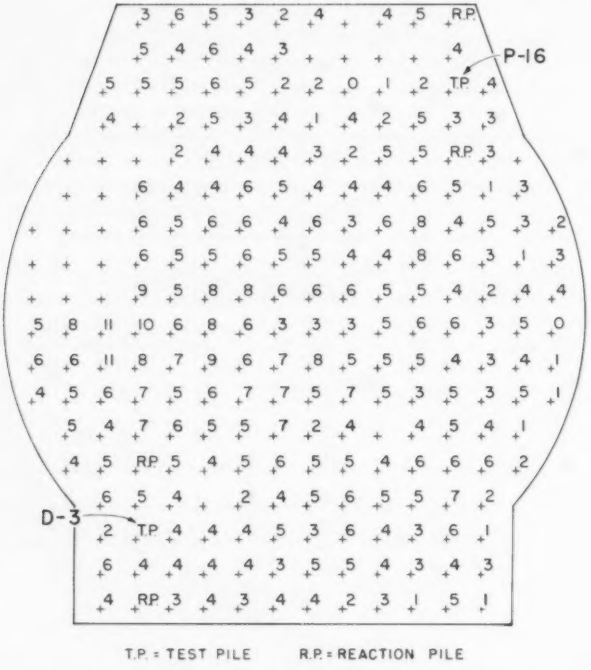

FIG. 18.—BLAST FURNACE PILES TOTAL RECORDED HEAVE, IN INCHES

tion, a bench mark reference point located within the excavated area but farther removed from the pile driving showed no signs of movement. It was therefore apparent that volume displacement of the steel H-piles, driven at the close 3 ft 4 in. spacing, was causing the heaving. In addition, the driving sequence probably contributed to the heaving because the ideal pattern of driving outward from the center of the pile group could not be followed. Instead space limitations made it necessary to adopt a "horseshoe" pattern, thereby entrapping the displaced medium clay and forcing it to move in the direction of driving.

Measurements were made during the construction period. The results were startling. For example, at the blast furnace foundation, immediately after pile

P-4 was driven, driving commenced on pile P-3. These two piles were only 3 ft 4 in. apart. During the driving of P-3, which required only one hour, pile P-4 came up 2-3/4 in.

Pile heaving was found to vary over wide limits, depending on the sequence of driving and the locations of the individual piles. Sometimes the cutoff mark was not placed immediately after a pile was driven and therefore the initial elevation was not recorded. Nevertheless, heaving of as much as 11 in. was observed. The total recorded quantities are shown in Fig. 18 for the blast furnace area.

Individual footings were analyzed. In the case of footing D10, shown in Fig. 19, it was found that the quantity of initial movement varied according to the position of the pile in the group and the sequence of pile driving; that is, the center piles heaved the most. Even when all the piles in the footing were installed, subsequent driving at other footings as much as 40 ft away caused further movement, but of uniform magnitude.

From these observations it was concluded that the volume displaced was relatively large and that the zone of influence was of the order of 50 ft. In addition, there was a time effect associated with pile heaving that could amount to 10 days or more. Cases were observed in which there was no pile driving in the vicinity for several days or during shut-down periods and yet piles continued to rise.

It is of interest to compute the volume of displacement of the steel H-piles, each having an area of 26.2 sq in. Within the blast furnace area proper, considering only the upper 66 ft of medium clay, the total displacement is approximately 3,000 cu ft. Distributed over the area of the blast furnace, this corresponds to a uniform heave of approximately one foot.

There was also a tendency for the upper soil layers to compress between the flanges of the H-pile and drive down, thus causing additional displacement. This was apparent from the fact that a 5 ft to 15 ft deep "hole," extending down from the surface between the flanges, was created by many of the driven piles. The volume loss could be attributed in part to compaction of the upper loose fill, but there is no doubt that some displacement of the underlying soils occurred. It has often been observed that when an H-pile is extracted from a silty-clay, a "slug" of soil adheres between the flanges. The friction of the soil and steel on three sides could be greater than the shear resistance of the soil itself on the fourth side. In such instances a supposedly non-displacement pile can displace considerable volume by carrying down material from upper layers.

The practical significance is obvious. Since the upper medium clay layer is being displaced upward, it tends to pull piles out of the hard clay layer. It is therefore essential that splices develop adequate strength in uplift, despite the fact that the piles will eventually carry only compression loads. In addition, where a pile is end-bearing, redriving should not commence until the heaving has stabilized. Because of the time effect, it may be necessary to wait several weeks.

Re-driving.—The pile tips were founded in the stiff clay layer and, because some of the load would be eventually transferred to this lower stratum, it was decided to redrive. The procedure took only three days and was relatively inexpensive.

All piles that had heaved more than 1 in. were driven 6 in. plus the quantity of known rise. On this basis redriving was restricted to the piles in the re-

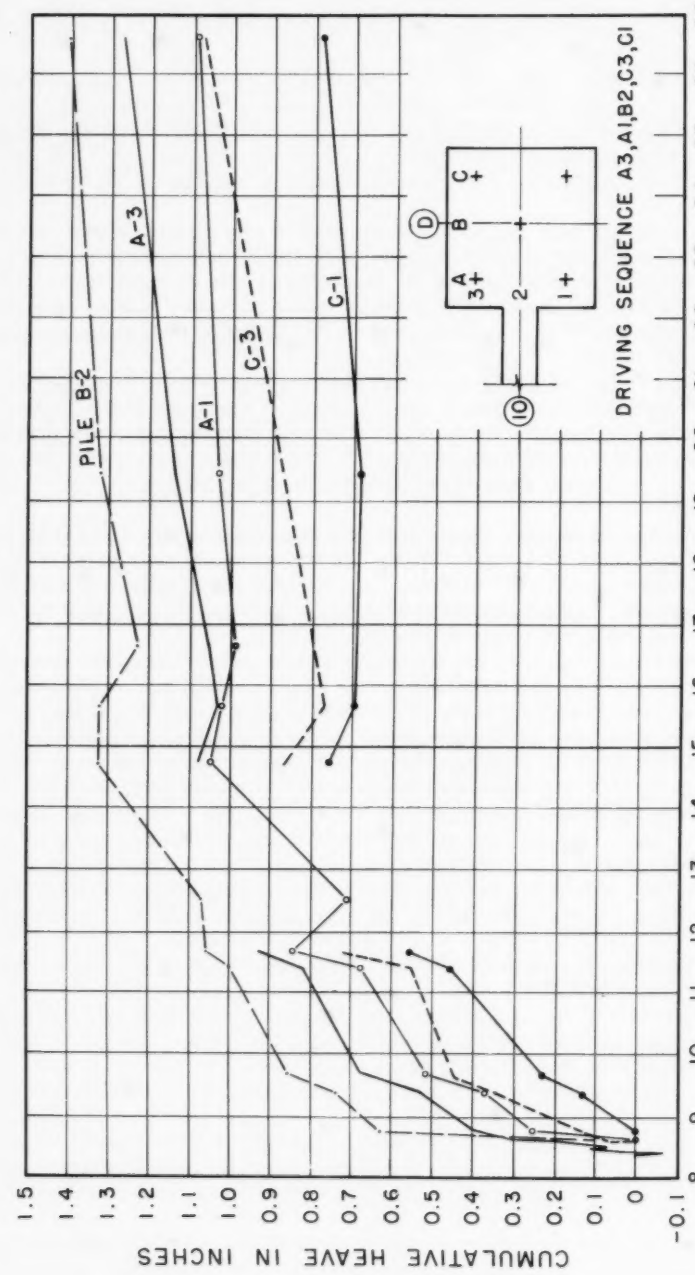

FIG. 19.—PILE HEAVING—FOOTING D-10

gion of the blast furnace. The piles driving for the trestle structure were spaced further apart; the heaving was less than 1 in. and therefore they were not redriven.

Redriving of the piles led to another series of interesting observations. The total number of blows and the distances redriven were recorded for 280 piles. The average blows per inch were plotted against the number of days between the initial driving and the redriving. The relationship is shown in Fig. 20. The results are erratic, due in part to the difficulty of operating the hammer uniformly for each redriven pile. Nevertheless there is a distinct trend of increase in redriving resistance with time, even up to 43 days.

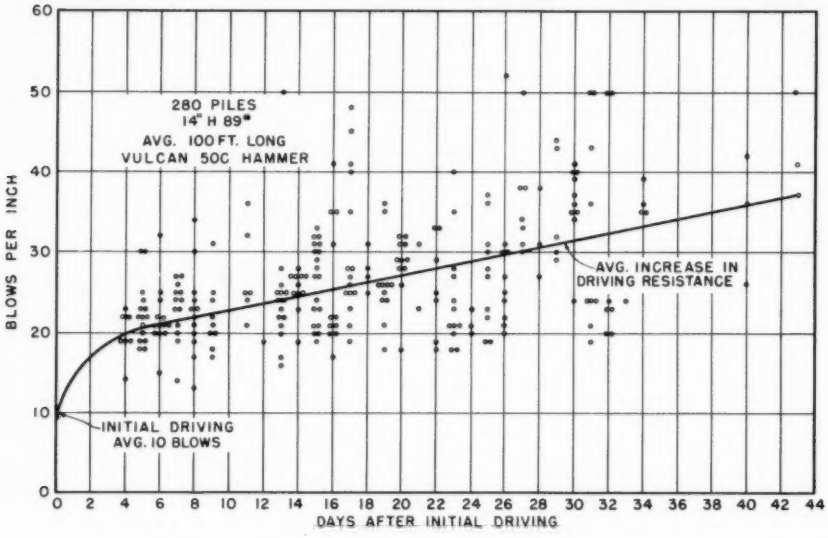

FIG. 20.—REDRIVING OF PILES

Conclusions.—The observations made permit the following conclusions with respect to steel H-piles on clay soils:

1. The load-carrying capacity of friction piles is not decreased because of pile heaving. A decrease would be expected only where the pile derives its ultimate support by end bearing.
2. Long steel H-piles driven close together and in large numbers have a sufficient cumulative displacement volume to cause pile heaving.
3. Splices must develop adequate strength to resist differential upward friction forces caused by an increase in soil volume displacement towards the surface.
4. Pile heaving continues long after driving has ceased (10 days), as the stresses in the soil readjust themselves. Since driving resistance increases

with time (doubled in four days), there is the problem of damaging the piles later when redriving.

The observations made by Klohn and the writer show that with minimal effort and expense useful engineering data can be obtained in the field.

Acknowledgments.—The writer expresses his appreciation to G. L. Berry, Chief Engineer of Jones & Laughlin Steel Corporation, and to D. L. Calhoun, Engineer at the Cleveland Works, for their helpful comments and permission to publish this discussion.

The blast furnace was designed by Arthur G. McKee and Company, Cleveland, Ohio. The piles were driven by the Equipment & Supply Company of Pittsburgh, Pa. The writer was employed at the time by Frederic R. Harris, Inc., Consulting Engineers. of which E, J. Quirin, F. ASCE, is the president and chairman.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Aero-Space Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU); and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 3029 is identified as 3029(ST8) which indicates that the paper is contained in the eighth issue of the Journal of the Structural Division during 1961.

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DECEMBER: 2668(ST12), 2669(IR4), 2670(SM6), 2671(IR4), 2672(IR4), 2673(IR4), 2674(ST12), 2675(EM6), 2676(IR4), 2677(HW4), 2678(ST12), 2679(EM6), 2680(ST12), 2681(SM6), 2682(IR4), 2683(SM6), 2684(SM6), 2685(IR4), 2686(EM6), 2687(EM6), 2688(EM6), 2689(EM6), 2690(EM6), 2691(EM6)[c], 2692(ST12), 2693 (ST12), 2694(HW4)[c], 2695(IR4)[c], 2696(SM6)[c], 2697(ST12)[c].

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JANUARY: 2698(PP1), 2699(PP1), 2700(HY1), 2701(SA1), 2702(SU1), 2703(ST1), 2704(ST1), 2705(SU1), 2706 (HY1), 2707(HY1), 2708(HY1), 2709(PO1), 2710(HY1), 2711(HY1), 2712(ST1), 2713(HY1), 2714(PO1), 2715 (ST1), 2716(HY1), 2717(SA1), 2718(SA1), 2719(SU1)[c], 2720(SA1)[c], 2721(ST1), 2722(PP1)[c], 2723(PO1)[c], 2724(HY1)[c], 2725(ST1)[c].

FEBRUARY: 2726(WW1), 2727(EM1), 2728(EM1), 2729(WW1), 2730(WW1), 2731(EM1), 2732(SM1), 2733(WW1), 2734(SM1), 2735(EM1), 2736(EM1), 2737(PL1), 2738(PL1), 2739(PL1), 2740(PL1), 2741(EM1), 2742(ST2), 2743(EM1), 2744(WW1), 2745(WW1), 2746(SM1), 2747(WW1), 2748(EM1), 2749(WW1), 2750(WW1)[c], 2751 (EM1)[c], 2752(SM1)[c], 2753(PL1)[c], 2754(ST2)[c], 2755(PL1).

MARCH: 2756(HY2), 2757(IR1), 2758(AT1), 2759(CO1), 2760(HY2), 2761(IR1), 2762(IR1), 2763(HY2), 2764 (ST3), 2765(HY2), 2766(HW1), 2767(SA2), 2768(CO1), 2769(IR1), 2770(HY2), 2771(SA2), 2772(HY2), 2773 (CO1), 2774(AT1), 2775(IR1), 2776(HY2), 2777(HY2), 2778(SA2), 2779(ST3), 2780(HY2), 2781(HY2)[c], 2782 (HW1)[c], 2783(SA2)[c], 2784(CO1), 2785(CO1)[c], 2786(IR1)[c], 2787(ST3)[c], 2788(AT1)[c], 2789(HW1).

APRIL: 2790(EM2), 2791(SM2), 2792(SM2), 2793(SM2), 2794(SM2), 2795(SM2), 2796(SM2), 2797(SM2), 2798 (EM2), 2799(EM2), 2800(EM2), 2801(EM2), 2802(ST4), 2803(EM2)[c], 2804(SM2)[c], 2805(ST4)[c].

MAY: 2806(SA3), 2807(WW2), 2808(HY3), 2809(WW2), 2810(HY3), 2811(WW2), 2812(HY3), 2813(WW2), 2814 (HY3), 2815(WW2), 2816(HY3), 2817(HY3), 2818(SA3), 2819(WW2), 2820(SA3), 2821(WW2), 2822(WW2)[c], 2823(HY3), 2824(SA3), 2825(HY3), 2826(SA3)[c], 2827(HY3)[c].

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JULY: 2847(PO2), 2848(SU2), 2849(HY4), 2850(PO2), 2851(HY4), 2852(PO2), 2853(SU2), 2854(HY4), 2855 (PO2), 2856(PO2), 2857(PO2), 2858(SA4), 2859(SU2), 2860(SA4), 2861(PO2), 2862(SA4), 2863(HY4), 2864 (HY4), 2865(HY4), 2866(HY4), 2867(HY4), 2868(PO2)[c], 2869(SA4)[c], 2870(SU2)[c], 2871(HY4), 2872(HY4)[c], 2873(SU2), 2874(SA4).

AUGUST: 2875(WW3), 2876(WW3), 2877(WW3), 2878(SM4), 2879(ST6), 2880(EM4), 2881(SM4), 2882(EM4), 2883(WW3), 2884(EM4), 2885(SM4), 2886(WW3), 2887(EM4), 2888(WW3), 2889(AT2), 2890(AT2), 2891 (AT2), 2892(AT2), 2893(AT2), 2894(AT2), 2895(AT2), 2896(AT2), 2897(AT2), 2898(AT2), 2899(AT2), 2900 (AT2), 2901(AT2), 2902(SM4), 2903(ST6), 2904(ST6), 2905(SM4), 2906(ST6), 2907(EM4), 2908(ST6), 2909 (EM4), 2910(ST6), 2911(EM4), 2912(SM4), 2913(ST6), 2914(WW3)[c], 2915(ST6)[c], 2916(EM4)[c], 2917(SM4)[c].

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OCTOBER: 2950(PP2), 2951(PP2), 2952(PP2), 2953(ST7), 2954(SM5), 2955(ST7), 2956(ST7), 2957(ST7), 2958 (ST7), 2959(SM5), 2960(EM5), 2961(EM5), 2962(ST7), 2963(ST7), 2964(EM5), 2965(SM5), 2966(SM5), 2967 (ST7), 2968(ST7), 2969(ST7), 2970(ST7), 2971(SM5), 2972(SM5)[c], 2973(EM5)[c], 2974(ST7)[c], 2975(PP2).

NOVEMBER: 2976(HY6), 2977(HY6), 2978(HY6), 2979(PO3), 2980(WW4), 2981(HY6), 2982(HY6), 2983 (HY6), 2984(HY6), 2985(SA6), 2986(SA6), 2987(WW4), 2988(WW4), 2989(WW4), 2990(HY6), 2991 (WW4), 2992(HY6), 2993(PO3), 2994(WW4), 2995(PO3), 2996(HY6), 2997(SA6), 2998(PO3), 2999 (PO3)[c], 3000(HY6)[c], 3001(SA6), 3002(SA6), 3003(SA6)[c], 3004(WW4)[c].

DECEMBER: 3005(IR4), 3006(RI4), 3007(IR4), 3008(SM6), 3009(EM6), 3010(ST8), 3011(IR4), 3012(SM6), 3013(EM6), 3014(ST8), 3015(IR4), 3016(ST8), 3017(EM6), 3018(EM6), 3019(EM6), 3020(SM6), 3021(ST8), 3022(ST8), 3023(ST8), 3024(ST8), 3025(ST8), 3026(EM6), 3027(EM6), 3028(EM6), 3029(ST8)[c], 3030(EM6)[c], 3031(IR4)[c], 3032(SM6)[c].

c. Discussion of several papers, grouped by divisions.

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